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A COMPARISON OF DESIGN METHODS FOR AIRFIELD PAVEMENTS

PROGRESS REPORT OF THE COMMITTEE ON CORRELATION OF RUNWAY DESIGN PROCEDURES OF THE AIR TRANSPORT DIVISION

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

A COMPARISON OF DESIGN METHODS FOR AIRFIELD PAVEMENTS

PROGRESS REPORT OF THE COMMITTEE ON CORRELATION OF RUNWAY DESIGN PROCEDURES OF THE AIR TRANSPORT DIVISION

Introduction

The necessity for providing soundly designed and economical pavements for airfields during World War II stimulated wide interest among engineers acquainted with this field. Since the war, the subject of airport pavement design continued to receive much attention, largely because of the increase in weight of aircraft and the rapid expansion of military airfields.

A completely rational method for the design of airport pavements has not been developed. The methods in use (1952) are either entirely empirical or at best only partly rational. Therefore, the reliability of any method is dependent upon experience or upon experimental verification. Any one of the methods requires considerable judgment on the part of the engineer who applies it.

The committee has tried to describe briefly and to compare the methods used in the United States and Canada. The objective was not to pass judgment on the individual methods but to give the practicing engineer the background needed by him in making his own choice. These methods, as designated herein, are as follows: (1) The Civil Aeronautics Administration (CAA) method, (2) The Corps of Engineers (United States Department of the Army) or California Bearing Ratio (CBR) method, (3) The Department of Transport (Dominion of Canada) method, (4) The United States Department of the Navy (Bureau of Yards and Docks) method, and (5) The Westergaard method for the design of rigid pavements.

Methods 2, 3, and 4 are used exclusively for the design of flexible pavements (bituminous); the Westergaard method is used for rigid pavements (Portland cement concrete); and the CAA method is used for either flexible or rigid pavements. The CAA method is empirical and utilizes soil classification tests; the CBR method and the Canadian method are also empirical, and utilize the CBR test and a plate loading test, respectively, to evaluate the supporting capacity of the subgrade. The Navy method and the Westergaard

Note.—Written comments are invited for publication; the last discussion should be submitted by June 1, 1953.

method are based partly on theory and partly on experience, and utilize plate bearing tests.

DEFINITIONS

There are widespread differences in opinion as to the proper definitions of the elements of the pavement structure. The definitions contained herein are not to be construed as being unanimously recommended by the committee. However, they have been adopted by some agencies.

For the purpose of this report, the committee has defined the various elements of the pavement structure as follows (see Fig. 1):

Pavement.—A structure consisting of one or more layers of processed materials. The layers are referred to as the wearing surface, the base course, and the subbase course.

Wearing Surface.—The upper, or surface, layer of a pavement is the wearing surface. The principal functions of the wearing surface are as follows: (1) It waterproofs the base against the penetration of surface water; (2) it protects the base against the disintegrating effects of traffic; (3) it provides a smooth riding surface; and (4) it distributes load to the underlying layer of the pavement.

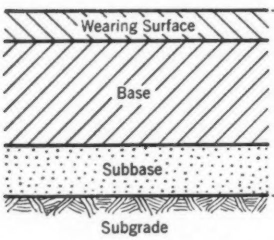

FIG. 1.—STRUCTURAL ELEMENTS OF A PAVEMENT

Base Course.—A layer of processed material directly underneath the wearing surface is called the base course. Its function is to distribute the load to underlying layers of the pavement structure, or to the subgrade when a subbase is not placed. Base courses may consist of untreated crushed stone, gravel, soil, or granular materials, mixed with binders such as asphalt or Portland cement.

Subbase Course.—A layer of material placed beneath the base course is the subbase course. The function of the subbase course is similar to that of the base course. It is usually composed of locally available, unprocessed granular material having a higher supporting value than that of the subgrade upon which is placed, but lower than that of the base course. Subbases are used in localities where thick pavements are required.

Subgrade.—The natural soil on which the pavement rests—whether embankment or excavation—is termed the subgrade.

A pavement is usually referred to as being "flexible" when the wearing surface is composed of a mixture of bituminous material and aggregate and "rigid" if Portland cement concrete is used. Normally, a flexible pavement is made up of a wearing surface composed of a bituminous aggregate mixture and a base course (either with or without a subbase) composed of crushed stone, slag, or gravel aggregates having the gradation and characteristics necessary to produce satisfactory mechanical stability.

A Portland cement concrete pavement is usually made up of a concrete slab resting on a layer of locally available granular material. Many agencies call this layer a subbase course, but, according to the definitions stated herein, it would be termed a base course.

The CAA Method

The CAA has developed a method of pavement design that is substantially a comparison of local conditions with statistical analyses of soil, drainage, frost, and loading conditions for many actual behavior patterns of airports in service. The method is based on a special soils classification, developed by the CAA, according to the characteristics listed subsequently under the heading, "Mechanical Analysis" in Table 2.

Soil classification by this system requires only a mechanical analysis and the determination of the Atterberg liquid and plastic limits. However, the CAA indicates the desirability of performing additional classification tests in some instances to permit a better evaluation of the probable performance of some soils.

Design loads are selected according to the type of air carrier service rendered. The pavement loading data given in Table 1 therefore represent the

TABLE 1.—Design Wheel Loads

Airport class	Air carrier service	Pavement Loading, in Kips	
		Single wheel	Dual wheels
3	Feeder	15	20
4	Trunk line	30	40
5	Express	45	60
6	Continental	60	80
7	Intercontinental	75	100
8	Intercontinental express	100	125

design load for each airport class except class 1 and class 2—the personal and secondary type airports. By design convention, the static gross plane load is equally divided between the two main wheel assemblies. Thus, the design aircraft for a class 6 airport is 120 kips gross plane load for an airplane equipped with single-wheel landing gear, or 160 kips gross plane load for an airplane equipped with dual-wheel assemblies. In either case, the design single-wheel load is 60 kips.

The soils classification system used in the CAA pavement design method is outlined in Table 2. The principal division is made at the class having 45% passing the No. 270 sieve (considering only the fraction that passed the No. 10 sieve). Soils composed of less than 45% combined silt and clay are classed as granular and identified as soils groups E–1 through E–5.

The detailed descriptions of the soil groups are given in the following paragraphs. These descriptions are quoted from "Airport Paving," published by the CAA.[1]

Soil Group Descriptions.—Group E–1 includes well-graded, coarse, granular soils that are stable even under poor drainage conditions and are not subject to detrimental heaving caused by frost. Soils of this group may conform to requirements for soil-type base courses such as well-graded sand clays with excellent binder. Group E-2 is similar to group E-1; but soils of this group

[1] "Airport Paving," Civ. Aeronautics Administration, Dept. of Commerce, Washington, D. C., May, 1948.

have less coarse sand and may contain greater percentages of silt and clay. Consequently, they may become unstable when poorly drained, and they are subject to a limited amount of heaving caused by frost.

Groups E-3 and E-4 include the fine, sandy soils of inferior grading. They may consist of fine cohesionless sand or sand-clay types having a fair to good quality of binder. They are less stable than group E-2 soils when submitted to adverse conditions of drainage and frost action.

Group E-5 comprises all poorly-graded granular soils having more than 35% and less than 45% of silt and clay combined. Also, this group includes all soils containing less than 45% silt and clay having plasticity indices greater

TABLE 2.—SOILS CLASSIFICATION SYSTEM FOR USE IN THE CAA PAVEMENT DESIGN METHOD

Soil group[a]	Mechanical Analysis, in Percentages						Subgrade Classes[b]							
							Flexible Pavement				Rigid Pavement			
							Drainage				Drainage			
							Good		Poor		Good		Poor	
	Retained on No. 10 sieve	Coarse sand passing No. 10 retained on No. 60	Find sand passing No. 60 retained on No. 270	Combined silt and clay, passing No. 270	Liquid limit	Plasticity index	No frost	Severe frost	No frost	Severe frost	No frost	Severe frost	No frost	Severe frost
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
E-1	0 to 45	40+	60−	15−	25−	6−	F a	F a	F a	F a	R_1 a	R_1 a	R_1 a	R_1 a
E-2	0 to 45	15+	85−	25−	25−	6−	F a	F a	F_1	F_2	R_1 a	R_1 a	R_1 a	R_1 a
E-3	0 to 45	0	0	25−	25−	6−	F_1	F_1	F_2	F_2	R_1 a	R_1 a	R_1 a	R_1 a
E-4	0 to 45	0	0	35−	35−	10−	F_1	F_1	F_2	F_3	R_1 a	R_1 a	R_1 a	R_2 a
E-5	0 to 45	0	0	45−	40−	15−	F_1	F_2	F_3	F_4	R_1 a	R_1 b	R_1 b	R_2 b
E-6	0 to 55	0	0	45+	40−	10−	F_2	F_3	F_4	F_6	R_1 a	R_2 b	R_2 b	R_2 b
E-7	0 to 55	0	0	45+	50−	10 to 30	F_3	F_4	F_6	F_7	R_1 b	R_2 b	R_2 b	R_2 c
E-8	0 to 55	0	0	45+	60−	15 to 40	F_4	F_6	F_7	F_8	R_1 b	R_2 c	R_2 c	R_2 d
E-9	0 to 55	0	0	45+	40+	30−	F_5	F_7	F_7	F_9	R_2 b	R_2 c	R_2 c	R_2 d
E-10	0 to 55	0	0	45+	70−	20 to 50	F_5	F_7	F_8	F_9	R_2 b	R_2 c	R_2 c	R_2 d
E-11	0 to 55	0	0	45+	80−	30+	F_6	F_8	F_9	F_{10}	R_2 c	R_2 d	R_2 d	R_2 e
E-12	0 to 55	0	0	45+	80+	....	F_8	F_9	F_{10}	F_{10}	R_3 d	R_2 e	R_2 e	R_2 e

[a] Group E-13 is composed of muck and peat, identified by field examination and not suitable for subgrade. [b] For explanation of symbols, see text and subsequent illustrations.

than 10. A plasticity index greater than 15 would cause a soil to be classified with the fine-grained soils, even though it contained more than 55% sand.

The E-6 group consists of the silts and silty loam soils having plasticity indices that are low or zero. These soils are friable and quite stable when dry or when at low moisture contents. They lose stability and become very spongy when wet, and for this reason they are difficult to compact unless the moisture content is controlled carefully. Capillary rise in the soils of this group is very rapid, and they are subject to detrimental heaving caused by frost more than soils of any of the other groups.

Group E-7 includes the clay loams, silty clays, and some sandy clays. When dry, these soils have consistencies that range from friable to hard, and

they are plastic when wet. These soils are stiff and dense when compacted at the proper moisture content. Variations in moisture are apt to produce detrimental volume changes. Capillary forces acting in the soil are strong, but the rate of capillary rise is relatively slow, and frost heave—while detrimental—is not as severe as it is in the soils of group E-6.

The soils of group E-8 are similar to those of group E-7, but the higher liquid limits indicate greater compressibility, expansion, and shrinkage, and also lower stability under adverse moisture conditions.

Group E-9 comprises the silts and clays containing micaceous and diatomaceous materials. These soils are highly elastic and unusually difficult to compact. They have low stability in both the wet and dry states, and they are subject to frost heave.

Group E-10 includes the silty clay and clay soils that form hard clods when dry and are plastic when wet. They have high compression indices, possess the ability to expand or shrink readily, are highly elastic, and are subject to frost heave. Soils of this group are more difficult to compact than are those of groups E-7 and E-8, and they require careful control of moisture in order to produce a dense, stable fill.

Group E-11 soils are similar to those of group E-10, but they have higher liquid limits. This group includes all soils with liquid limits between 70 and 80 and with plasticity indices over 30.

Group E-12 comprises all soils having liquid limits over 80, regardless of their plasticity indices. These soils may be highly plastic clays that are extremely unstable in the presence of moisture, or they may be highly elastic soils containing mica, diatoms, or organic matter in excessive percentages. Whatever the cause of their instability may be they will require the maximum corrective measures.

Group E-13 includes organic swamp soils—such as muck and peat—that are recognized by examination in the field. They are characterized by their lack of stability and unusually low density in their natural state, and by high moisture content.

Certain fine-grained soils might be classed in more than one group if the criteria given in Table 2 were the only ones availagle for classification. Soils containing mica, diatoms, or considerable colloidal material, and those exhibiting a plasticity index that is greater than the maximum plasticity index corresponding to the maximum liquid limit of the particular group may be classified[1] by the application of Fig. 2.

At the discretion of the designer, upgrading of the soil rating one or two classes may be permitted when the fraction of the total sample retained on the No. 10 sieve exceeds 45% for groups E-1 through E-5. Similar upgrading may be permitted for other groups if at least 55% of the total sample is retained on the No. 10 sieve. In permitting such upgrading, the decision of the engineer should be influenced by the quality and grading of the coarse material. (For each soil group, Table 2 indicates the subgrade classes that represent the performance of the group as subgrade for flexible or rigid pavement.)

Pavement Design.—Fig. 3 is a design chart for rigid pavements.[1] It is to be entered with the single-wheel load and subgrade class as arguments. The

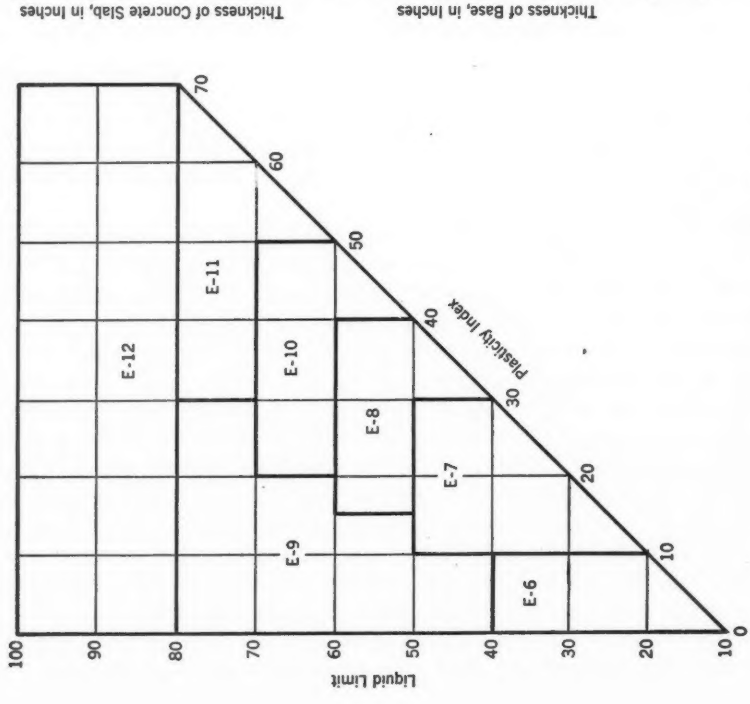

Fig. 2.—Classification Chart for Fine-Grained Soils

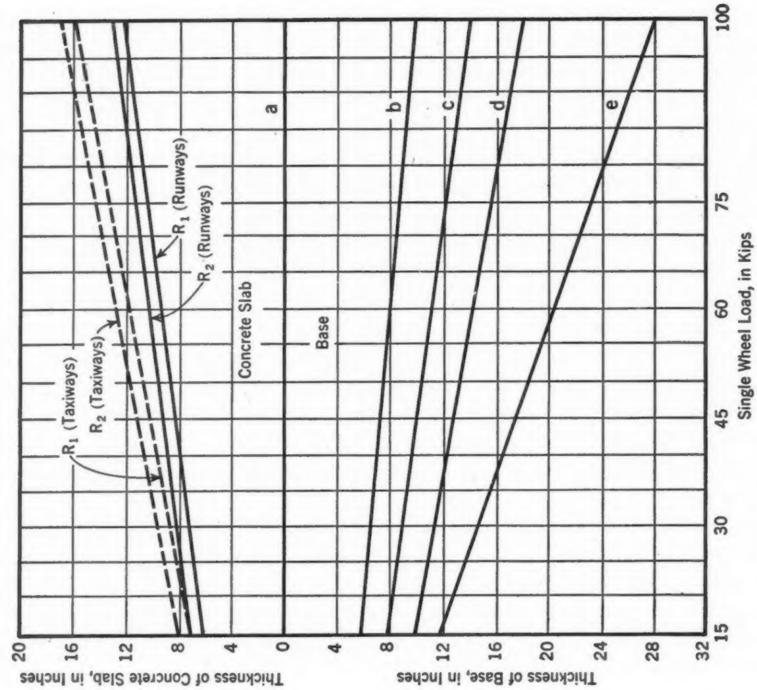

Fig. 3.—Concrete Pavement Design Chart, CAA Method

zone above the zero line gives the necessary thickness of Portland cement concrete slab. The required thickness of slab is the difference in ordinate between the zero line and the line R_1 or R_2, depending on the subgrade. The dashed curves give values applying to taxiways, aprons, and runway ends, and the solid curves give values for runways.

The lower zone of the chart gives the thickness of base necessary for placing between the subgrade and the concrete slab. Again, the chart is entered with the single-wheel load. The difference in ordinate between the zero line and line a, b, c, d, or e, depending on subgrade class, will indicate the base requirement. These values apply to the base beneath runways and beneath taxiways, aprons, and runway ends.

In 1950, the CAA[2] initiated a coordinated pavement condition survey with the broad objective of studying the performance of airport pavements in order to check the soundness of the design standards that had been established in the publication "Airport Paving."[1] As a result of this survey, certain minor changes can be made in the design charts for rigid pavements. These changes are as follows:

(1) The curves designated R_2 (runways) and R_2 (taxiways) can be eliminated.

(2) The taxiway requirement for a wheel loading of 15 kips may be reduced to 6 in., and for the 100-kip loading it may be lowered to 15 in.

As an example of the design procedure for Portland cement concrete surfaces, consider a taxiway designed to support a gross plane load of 150 kips. The soil investigation results place the subgrade in group E-7. The site is so located that frost penetration is not a design consideration, and drainage is good. Table 2 indicates that the subgrade classification for a rigid-type wearing surface is R_1 b. Fig. 3 is entered with a wheel load of 75 kips. The necessary slab thickness is seen to be 13 in., and the required depth of the granular base is 9 in.

On the basis of experience with local conditions, certain adjustments may be made in the subbase thickness obtained from the design charts. For instance, reductions in thickness of up to 35% may be made for pavements in arid regions where the local behavior patterns of soils and pavements warrant such reduction. Regardless of load or subgrade, the minimum thickness of 6 in. of Portland cement concrete must be used.

In some localities, conditions accompanying frost heave may require increasing the base thickness obtained from the design charts. However, since concrete pavements may possess an insulating property that acts to prevent frost penetration, the frost penetration for certain localities may be reduced by as much as one half the thickness of the concrete slab.

Figs. 4, 5, 6, and 7 are the charts for the design of flexible pavements. As with the Portland cement concrete pavements, differentiation is made between pavements for runways and pavements for taxiways, aprons, and runway ends. The dotted lines represent the recommendation of the condition survey

[2] "Airport Pavement Performance," *Airport Engineering Bulletin No. 2*, Civ. Aeronautics Administration, Dept. of Commerce, Washington, D. C., April, 1952.

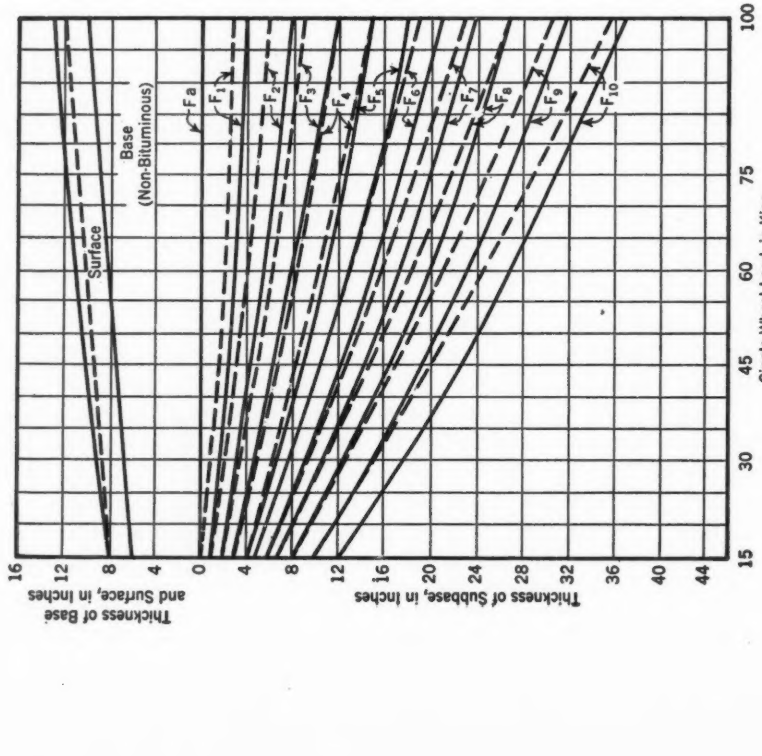

FIG. 4.—FLEXIBLE PAVEMENT, RUNWAYS, NON-BITUMINOUS BASE, CAA METHOD

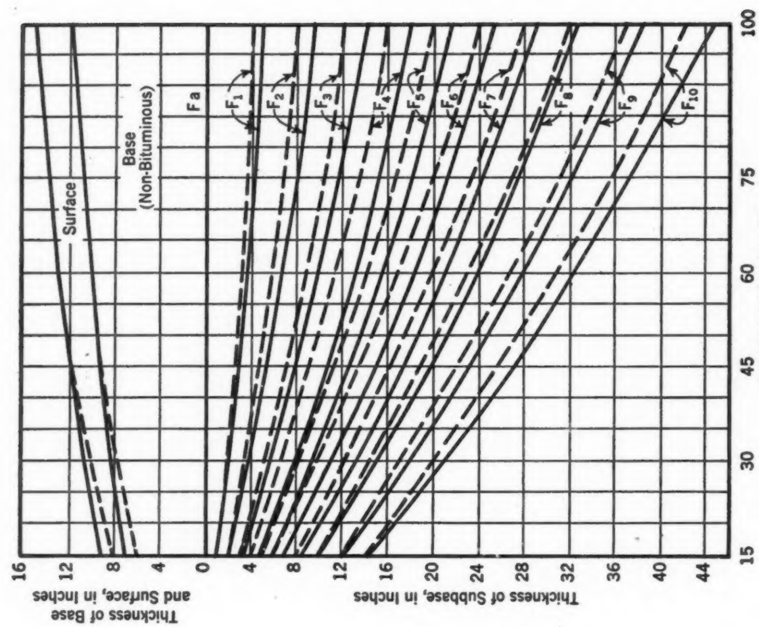

FIG. 5.—FLEXIBLE PAVEMENT FOR TAXIWAYS, APRONS, AND RUNWAY ENDS; NON-BITUMINOUS BASE, CAA METHOD

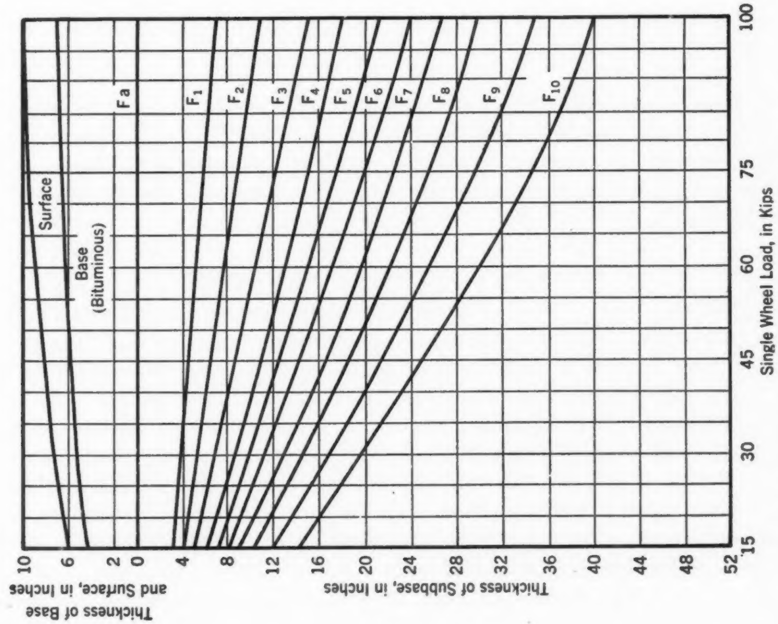

FIG. 6.—FLEXIBLE PAVEMENT, RUNWAYS, BITUMINOUS BASE, CAA METHOD

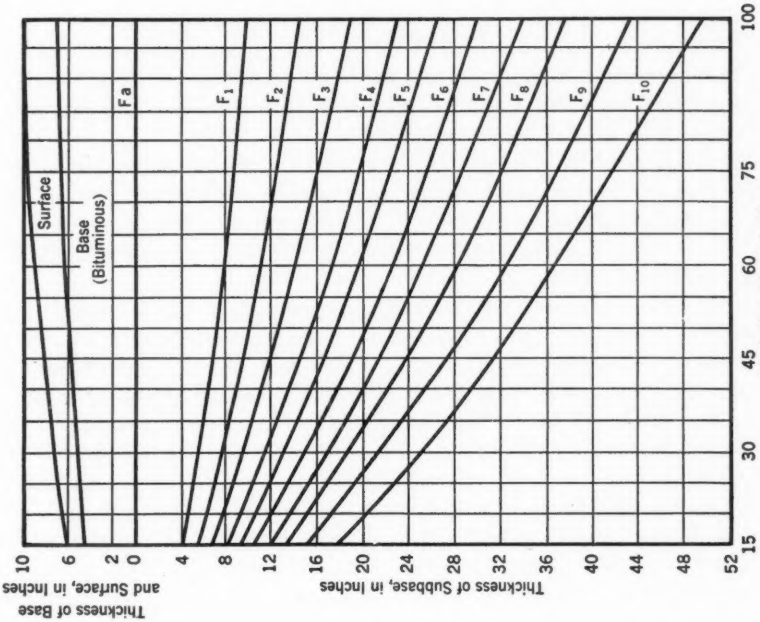

FIG. 7.—FLEXIBLE PAVEMENT FOR TAXIWAYS, APRONS, AND RUNWAY ENDS; BITUMINOUS BASE, CAA METHOD

of 1950.[2] It is understood that the dotted lines will replace the solid lines in the new *Airport Paving Manual* scheduled for publication in 1953.

There is a further distinction made for flexible pavements in that charts have been prepared for wearing surfaces on bases of untreated mineral aggregate (Figs. 4 and 5)[2] and for wearing surfaces to be placed on plant-mix bituminous-treated base courses (Figs. 6 and 7).[1]

The information required in the application of the design charts for flexible pavements consists of the single-wheel load and the subgrade class. The subgrade class is obtained from Table 2. The single-wheel load should correspond to the class of air service for which the facility is designed. These classes are listed in Table 1.

The thickness of wearing surface is the difference in ordinate between the two uppermost curves on the design chart. The required base thickness is the difference in ordinate between the zero line and the curve denoting the interface between the surface and the base. The necessary depth of subbase is given by the length of the ordinate from the zero curve down to one of the curves marked F_1, F_2, F_3 . . . F_{10}—the subgrade obtained from Table 2.

As an example of the procedure, consider the design of a flexible pavement runway to carry a single-wheel load of 60 kips. Analysis of the subgrade soil group results in its being classified E-4. The airport site is in a region where frost need not be considered in the design but where drainage is poor. From Table 2, the subgrade classification is F_2. The base course is to be constructed of an untreated mineral aggregate. Therefore, the proper design chart is Fig. 4 (dotted lines). The wearing-surface requirement is 2 in. of a high-grade type of bituminous mixture; the necessary thickness of base course is 8 in. (to the nearest inch); and the depth of selected subbase material is 4 in. (to the nearest inch).

With light wheel loads, the total thickness of wearing surface, base, and subbase will sometimes be found insufficient to satisfy the requirements for resisting frost action. In these cases, the necessary increase in thickness over the structural requirement may be determined by increasing the depth of the selected subbase.

The Corps of Engineers (CBR) Method

The CBR test and method of design for highways were developed by the California Division of Highways under the general supervision of T. E. Stanton, M. ASCE, then materials and research engineer, and under the direction of O. J. Porter, M. ASCE, who was senior physical testing engineer. At the beginning of World War II, the Corps of Engineers, after a survey of the flexible pavement design methods available, adopted the empirical method then in use by the California Division of Highways known as the CBR method.[3]

Application of the CBR method enables the designer to determine the required thicknesses of subbase, base, and wearing surface by entering a set of design curves with the results of soil tests. The California Division of Highways developed design curves for 7-kip and 13-kip wheel loads but the large

[3] "Development of CBR Flexible Pavement Design Method for Airfields: a Symposium," *Transactions*, ASCE, Vol. 115, 1950, p. 453.

wheel loading of aircraft made it necessary to conduct studies in order to extend the design curves for airfield design. These studies resulted in the airfield design curves included in subsequent pages of this report. Also presented are data for the determination of the thicknesses of material necessary to prevent detrimental frost heave.

Soil Tests and Subgrade Characteristics.—The design procedure for application of the CBR method consists of the determination of a modulus of shearing resistance of the subgrade soil or base course material and the application of these data to empirical design curves from which are read the required thicknesses of the various courses. The soil penetration test is considered valid only when a large part of the deformation caused by load is shear deformation. The "California Bearing Ratio" is expressed as a percentage of the standard stability value for crushed stone. In areas where soils are adversely affected by frost action, considerations other than shear strength may control the design of the pavement.

For design purposes, the penetration test may be performed on samples that are remolded, on undisturbed samples secured from compacted embankments or excavations, or on the soil in place. Where compaction in the field is practicable, the subgrade soil should be tested in the compacted state, but where field compaction is impracticable it may be tested in the undisturbed condition. Because saturation under load is the worst possible condition for silty and clayey soils, the design test for these types of soil is conducted on a soaked sample.

TABLE 3.—MINIMUM THICKNESS OF BITUMINOUS WEARING SURFACE

Wheel assembly	Tire pressure, (lb per sq in.)	Load assembly, (kips)	Minimum thickness, (inches)
Single	100	up to 30	2
		over 30	3
	200	up to 30	3
		over 30	4
Dual[a]		up to 45	2
		45 to 65	3
		above 60	4
Twin tandem[b]		up to 125	3
		above 125	4

[a] Spacing, 37.5 in. [b] Spacing, 31 in. by 60 in., based on contact area of 267 sq in. per tire.

The design curves shown subsequently as Figs. 8 and 9 are based on CBR values at 0.1-in. penetration. These curves are consistent with data obtained from accelerated traffic tests and airfield pavement service records. The curve for each wheel load shows the total thickness of pavement (subbase, base, and wearing surface) required above a layer of material having a given CBR.

The base course immediately under the wearing surface should be sufficiently stable to withstand the high stresses produced in the zone directly under the wheel of an aircraft or highway vehicle. The required stability depends upon the type and thickness of the wearing surface, the magnitude of the loading, and the effect of a rolling or skidding wheel. It is desirable that the base course material underlying a bituminous wearing surface—especially

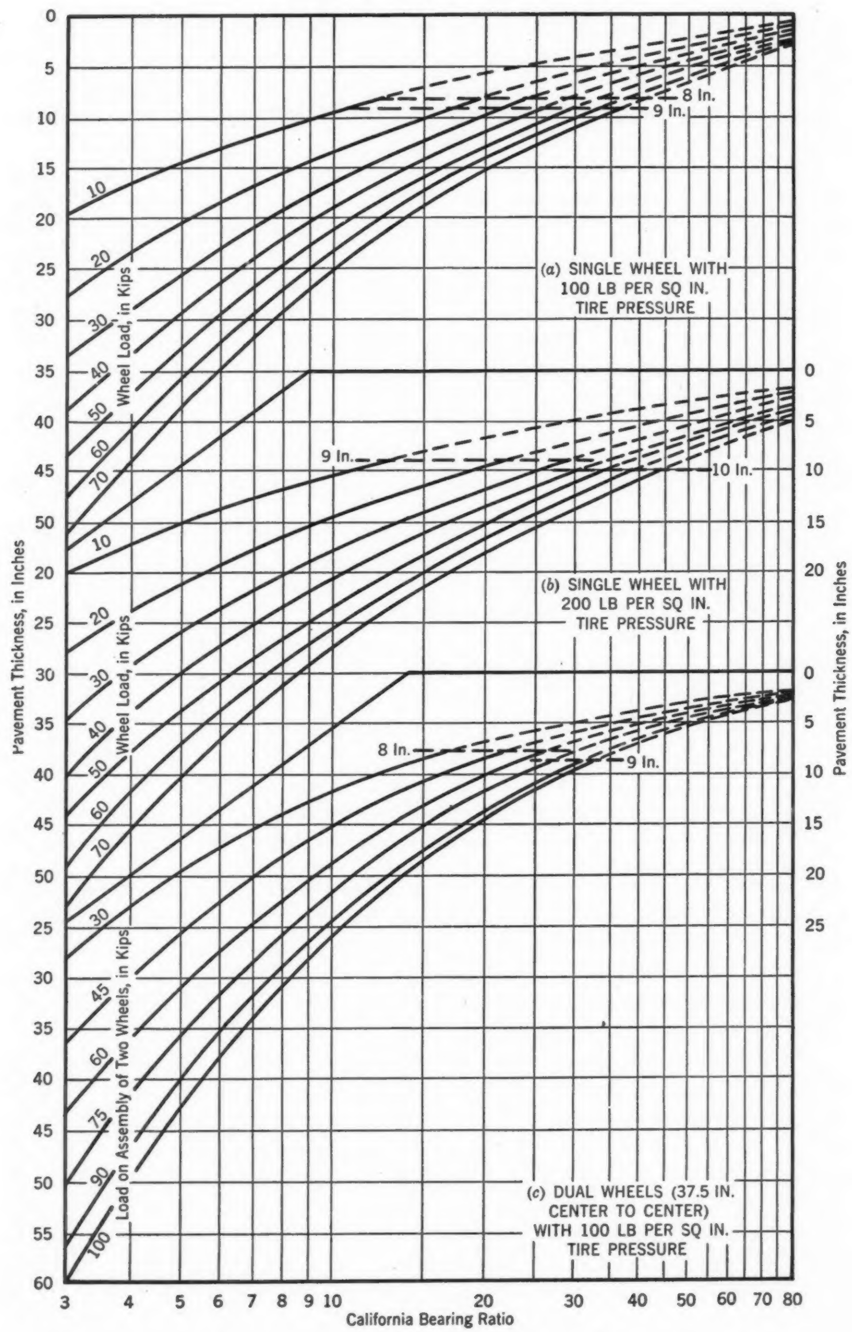

Fig. 8.—Flexible Pavement Design Curves—Constant Tire Pressure, CBR Method

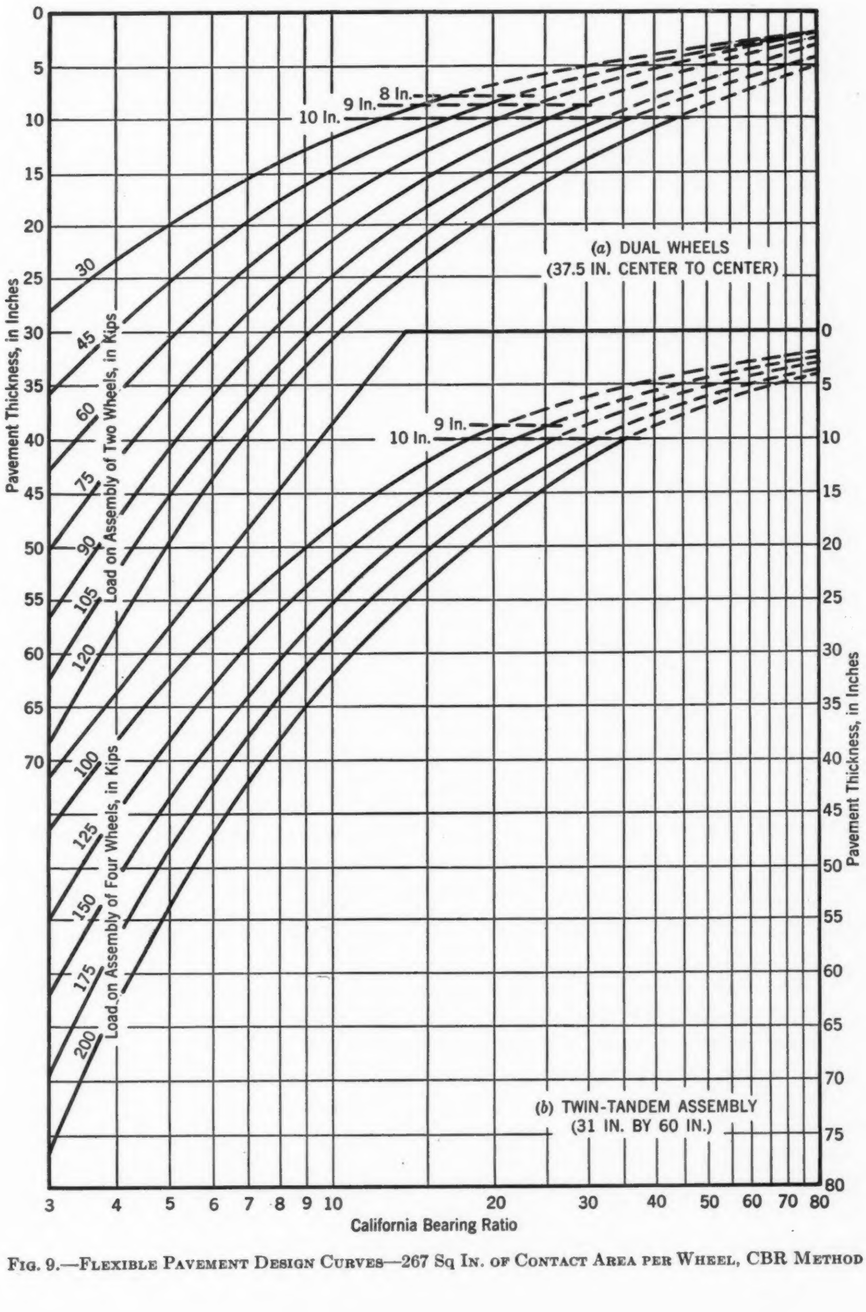

FIG. 9.—FLEXIBLE PAVEMENT DESIGN CURVES—267 SQ IN. OF CONTACT AREA PER WHEEL, CBR METHOD

for the heavier wheel loads—have a CBR of at least 80 and be at least 6 in. thick.

Bituminous wearing surfaces for airfields should be thick enough to prevent shearing displacement when subjected to the design wheel load. According to the recommendations of the Corps of Engineers, the thicknesses should be as shown in Table 3.[4]

Pavement Design.—Several design curves are shown. In these charts, the pavement thickness should be reduced by 10% for the central portions of runways (the area between two end sections of 1,000 ft each). The end sections and taxiways are designed by using the full thickness. Figs. 8(*a*) and 8(*b*) yield the required runway and taxiway pavement thickness for single-wheel loads of from 10 kips to 70 kips.[4] Fig. 8(*c*) indicates the pavement thickness required for a dual-wheel assembly similar to that of the B-50 bomber, in which the wheels are 37.5 in. center to center.[4] Fig. 9(*a*) indicates the pavement thickness required for the same type of wheel assembly, except that the con-

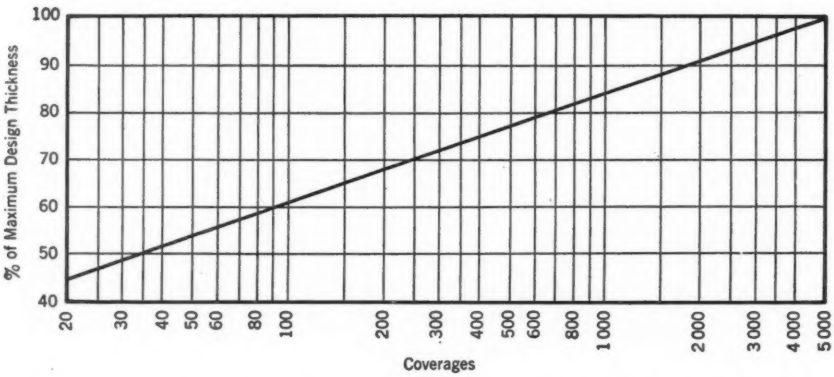

FIG. 10.—REDUCTION IN THICKNESS AS RELATED TO COVERAGES, FOR FLEXIBLE PAVEMENTS ONLY, CBR METHOD

tact area is kept constant instead of the tire pressure.[4] The pavement thickness required for twin-tandem assemblies similar to those on the B-36 bomber is given in Fig. 9(*b*).[4]

The design curves are based on capacity operation defined by the Corps of Engineers as 5,000 coverages, a coverage being dependent on cycles of operation and on tire size. If it is anticipated that the pavement will be subjected to less than 5,000 coverages during its economic life, a reduction in thickness is permitted. This reduction is indicated in Fig. 10.[4] The reductions are expressed as percentages of that maximum design thickness that applies to full operational and special airdromes. A "cycle of operation" is defined as one landing and one take-off. To obtain a coverage value, the number of cycles of operation should be divided by the appropriate factor for the type airplane to be used, selecting this factor as follows:

[4] "Airfield Pavement Design, Flexible Pavements," Engineering Manual for Military Construction, Corps of Engrs., Dept. of the Army, Washington, D. C., July, 1951, Part XII, Chapter 2.

Type of airplane	Factor
5-kip, single-wheel landing gear	25
15-kip, single-wheel landing gear	16
B-17 or B-24	12
B-29	6
B-36	4
B-50	8

To show the design procedure for use in cases in which the subgrade soil or base course materials are not affected by frost, or in which conditions requiring special treatment are not the principal considerations, an example is given.

Assume that a taxiway is to be designed for a wheel load of 50 kips at a tire pressure of 100 lb per sq in. The design CBR values of the compacted subgrade soil and of the materials available for base course construction are as follows:

Material	Design CBR
Compacted subgrade soil	8
Select material, subbase	20
Crusher run base	80

The total thickness of the pavement is governed by the CBR of the compacted subgrade soil. From Fig. 8(*a*) the required thickness is 25 in. The total thickness of 25 in. may be composed of the subbase, base, and wearing surface. Assume the desired wearing surface to be 4 in. thick. A pavement thickness of 13 in. is required over the layer of subbase material having a CBR of 20. The subbase will then be 12 in. thick and overlain with 9 in. of crusher run base. Since the minimum requirement of 6 in. of base course has been satisfied by this design, no further adjustment is necessary. If the central portion of a runway were to be designed for the same wheel load and tire pressure, the required pavement thickness would be 22.5 in., a decrease of 10%.

Frost Action.—In the preceding example, the action of frost has not been discussed. The strength of some soils is greatly reduced as a result of frost action, the reduction in strength, and the amount of frost heave depending on the type of soil, the temperature conditions during the freezing and thawing periods, the permeability of the soil, the level of the ground water, and the drainage conditions. Investigations have indicated that any soil that contains 3% or more, by weight, of grains smaller than 0.02 mm in diameter will be susceptible to objectionable frost action.[5]

A reliable indication of the effects of climate is the "freezing index," which is a measure of the combined duration and magnitude of below-freezing air temperatures occurring during any given winter. The normal freezing index is computed on the basis of air temperatures recorded over a long period of time. Fig. 11(*a*) indicates the method of determining the freezing index. A degree day is the algebraic difference between 32° F and the daily mean temperature. The degree day is positive when the daily mean temperature is below 32° F. The

[5] "Frost Conditions," Part XII of "Airfield Pavement Design," Office of the Chf. of Engrs., Corps of Engrs., Dept. of the Army, Washington, D. C., April, 1951, Chapter 4.

depth of pavement required to insulate the subgrade soil to prevent it from freezing is shown in Fig. 11(*b*). Heaving of the pavement will occur if the pavement that is provided is thin enough to permit freezing of the subgrade soil. The heaving will be uniform if soil and ground-water conditions are uniform, and will be irregular if soil and ground-water conditions are nonuniform. Under conditions that are conducive to irregular heaving, freezing of the subgrade soil should be prevented and Fig. 11(*b*) should be used for the design. If the

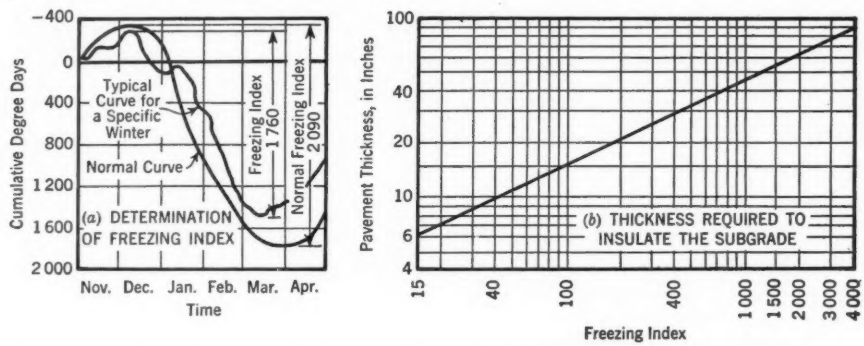

FIG. 11.—DESIGN OF PAVEMENTS SUBJECT TO DIFFERENTIAL FROST HEAVE, CBR METHOD

subgrade soils are not subject to differential heaving, less thickness of pavement than that required to prevent freezing is permissible. In such a case, the design is based on a reduction in the strength of the subgrade soil caused by frost action. Fig. 12[5] which reflects the reduction in strength of soil during thawing, should be used to determine the pavement thickness if limited frost action is permitted. The soils classification numbers by which these curves are designated are as follows:

F1.—Gravelly soils containing between 3% and 20% (by weight) material finer than 0.02 mm fall into this class.

F2.—Sands containing between 3% and 15% (by weight) material finer than 0.02 mm comprise this class.

F3.—This class is composed of (*a*) gravelly soils containing more than 20% (by weight) material finer than 0.02 mm and sands, except fine silty sands, containing more than 15% material finer than 0.02 mm and (*b*) clays with plasticity indices of less than 12, except varved clays.

F4.—This class is composed of (*a*) all silts, including sandy silts; (*b*) fine, silty sands containing more than 15% (by weight) material finer than 0.02 mm; (*c*) lean clays with plasticity indices of less than 12; and (*d*) varved clays.

When frost is permitted to penetrate group F4 soils, the same design curve should be used as for group F3 soils. Frost should be permitted to penetrate F4 soils only under flexible paved areas of lesser importance where appreciable nonuniform pavement heave may be tolerated. Frost should not be permitted to penetrate group F4 soils beneath rigid pavements.

In Fig. 12, the curves for use in the case of single-wheel landing gear (solid lines) apply when tire pressures are between 100 lb per sq in. and 200 lb per sq in., and the abscissas are the loads on a single wheel. The curves for the dual-wheel landing gear and for the twin-tandem assembly in Fig. 12 are based on a contact area of 267 sq in. per wheel, and each abscissa is the total load for one

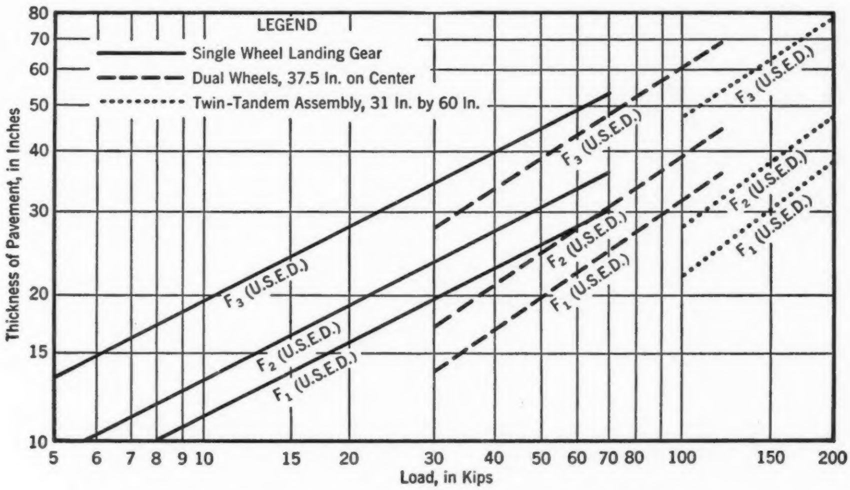

FIG. 12.—DESIGN CURVES FOR FLEXIBLE PAVEMENTS SUBJECTED TO FROST ACTION IN THE SUBGRADE SOIL

assembly. When using Fig. 12, the thickness should be reduced by 10% for the central portion of runways between the 1,000-ft end sections. The curves should be used without reduction in the design of runway ends, aprons, and taxiways. The value of pavement thickness obtained by the preceding method should be compared with the thickness determined by the regular procedure, and the greater value should be used.

THE DEPARTMENT OF TRANSPORT (CANADIAN) METHOD

In order to develop a method for flexible pavement design that would conform with Canadian experience, the engineers of the Department of Transport, Dominion of Canada, Ottawa, Ontario, initiated an extensive investigation of major airfields in Canada in the early spring of 1945. These studies and the subsequent development of the design method were conducted under the direction of Norman W. McLeod, engineering consultant to the Department of Transport.

Published data are available concerning the results obtained at the first thirteen airports tested.[6] These extend across Canada. At one of these, the subgrade consisted of about 80 ft of clean sand, and at another, from 3 ft to 5 ft of sand over clay. At the remainder, the subgrade was clay or clay loam.

[6] "Economical Flexible Pavement Design Developed from Canadian Runway Study," by Norman W. McLeod, *Engineering News-Record*, Vol. 142, April 28, May 12, May 26, and June 9, 1949.

All data were obtained from runways that had been in service for several years. To determine the supporting capacity of the subgrade, base course, and wearing surface at test locations on existing runways, repetitive load tests were made using steel bearing plates 13 in., 18 in., 24 in., 30 in., 36 in., and 42 in. in diameter.

Loads were applied in the plate bearing tests to give deflections of approximately 0.05 in., 0.2 in., and 0.5 in., so that the load test data could be used for the design of either rigid or flexible pavements. For flexible pavements, the critical deflection was considered to be 0.5 in. and for rigid pavements 0.05 in. The intermediate deflection of 0.2 in. was selected to provide a complete load deflection curve.

Results of Load Tests.—The first step in utilizing the data was to plot deflections against the number of repetitions of loading. Fig. 13 indicates the effect of load repetition on deflection (using a bearing plate having a 30-in.

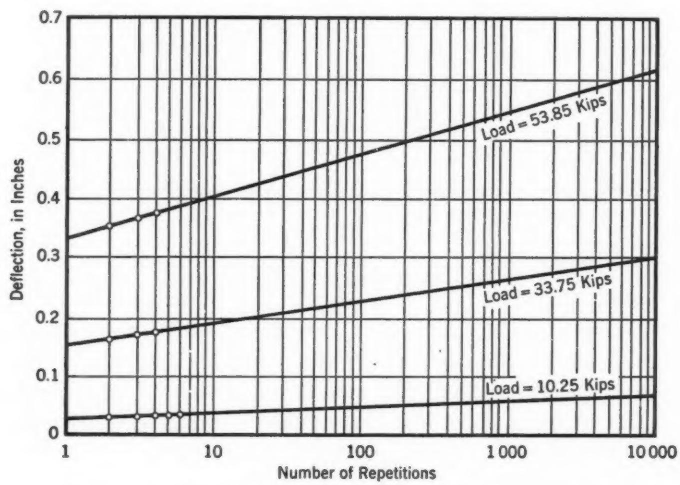

FIG. 13.—PAVEMENT DEFLECTIONS, SHOWING THE INFLUENCE OF REPETITION OF LOADING

diameter) for loads of three different magnitudes.[6] As a result of these studies, the Canadian Department of Transport adopted a 0.5-in. deflection for 10 repetitions of the applied load as the criterion for the design of flexible pavements for runways under conditions of capacity operations.

From graphs like Fig. 13, it was possible to draw the load-deflection curves for any number of repetitions of load. Study of the subgrade and surface load-deflection curves indicated that, for any given deflection, the load carried on a bearing plate of specified size at 1 repetition was about 115% of the load carried at 10 repetitions. The load carried at 100 repetitions was 89% of that supported at 10 repetitions; and the load carried at 1,000 repetitions was 80% of that at 10 repetitions.

There was a relationship between the subgrade support of a bearing plate of any size at a specified deflection (0.05 in. to 0.7 in.) and the subgrade support

of a plate having a diameter of 30 in. at a deflection of 0.2 in. A deflection of 0.2 in. was selected because it can be conveniently obtained with a bearing plate 30 in. in diameter. Using Fig. 14,[6] if the load causing a bearing plate 30 in. in diameter to deflect 0.2 in. is known, the loads that can be supported by bearing plates from 12 in. in diameter to 42 in. in diameter, producing deflections of from 0.05 in. to 0.7 in. may be computed. The ordinates in Fig. 14 represent the "subgrade support ratio," a ratio between the subgrade support for a specified bearing plate (whose deflection indicates the curve to be used) to the subgrade support beneath a bearing plate of 30-in. diameter when deflected 0.2 in. The lower scale gives the ratio of the perimeter to the bearing area of each plate. For example, assume that a pressure of 10 lb per sq in. is needed to deflect a plate 30 in. in diameter 0.2 in. What pressure will be

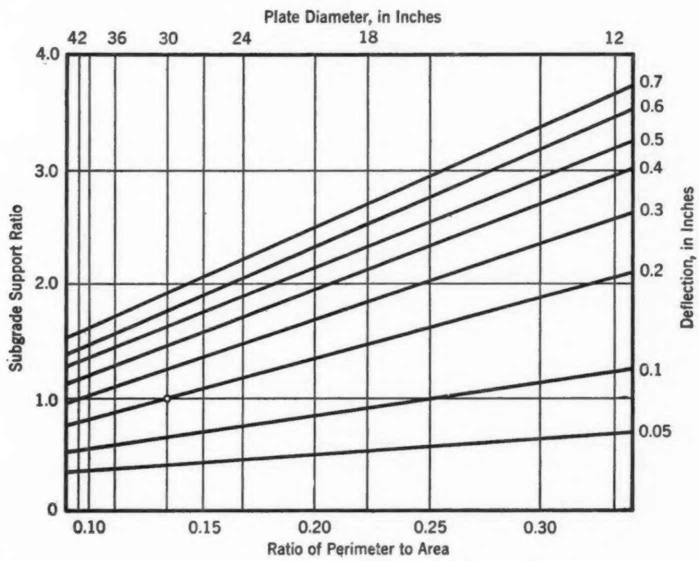

FIG. 14.—CURVES OF SUPPORT RATIOS FOR COHESIVE SUBGRADE SOILS AT TEN REPETITIONS of LOADING

required to deflect a plate 36 in. in diameter 0.5 in.? Fig. 14 is entered with the latter values, and indicates a subgrade support ratio of 1.45. By multiplying 10 lb per sq in. by this factor, the pressure for a plate 36 in. in diameter is found to be 14.5 lb per sq in.

Because it is costly and cumbersome to conduct load tests, North Dakota cone bearing tests, Housel penetrometer tests, CBR tests, and triaxial compression tests were made on the subgrade at each load test location. Reasonably good correlations were established between the results of each of these four tests and the plate bearing tests. These correlations appear subsequently in Fig. 17.

A graph was drawn of load tests made at the surface of a given thickness of granular base and wearing surface versus the load supporting value of the subgrade for the same deflection and bearing plate size. It was found that

the increase in the load carrying capacity provided by any given well-constructed pavement varies directly as the supporting value of its subgrade. From this important observation, Mr. McLeod has developed the following equation for determining the required thickness of flexible pavements:

$$T = K \log \frac{P}{S} \qquad (1)$$

in which T represents the required thickness of granular material, in inches; P is the gross wheel load, in pounds, to be carried by the pavement; S is the gross subgrade support, in pounds, for the same contact area as the wheel load and at the design criterion values for deflection and loading repetitions; and K designates the base course constant. Its value depends on the composition, moisture content, and density of the base course material. For very large thicknesses, its value must also depend on the depth of the base course.

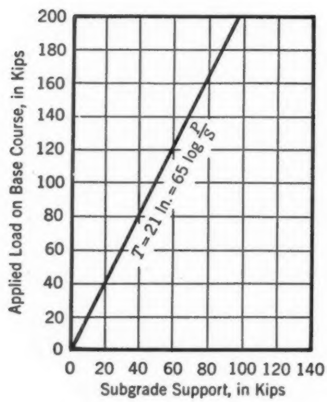

Fig. 15.—Variation of Deflection with Subgrade Support

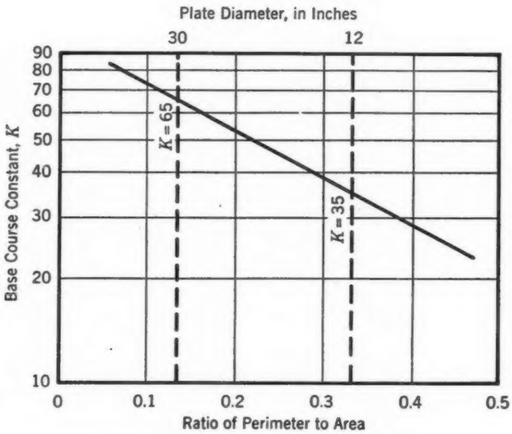

Fig. 16.—Relation of Plate Size to K-Value

The base course constant, K, represents an inverse measure of the supporting power of the base course, per unit thickness. To evaluate K, graphs of the relationship between applied load and subgrade support were prepared for thicknesses of 7 in., 14 in., and 21 in. of granular base course. Fig. 15 shows a typical curve for 21 in. of base course loaded by a bearing plate 30 in. in diameter—through 10 repetitions of loading—to produce a deflection of 0.5 in.[6] The slope of the straight line through the data for these three thicknesses of granular base gives a K-value of 65 for this plate. Similar data give a K-value of 35 for the 12-in.-diameter bearing plate. From a study of the available data, a relationship was established between the value of K and the size of the bearing plate,[6] which is shown in Fig. 16.

Design Procedure.—A set of design curves that give the required thicknesses of granular base for capacity operation of runways for a wide range of aircraft-wheel loadings is given in Fig. 17.[6] The curves are based on Eq. 1 using the

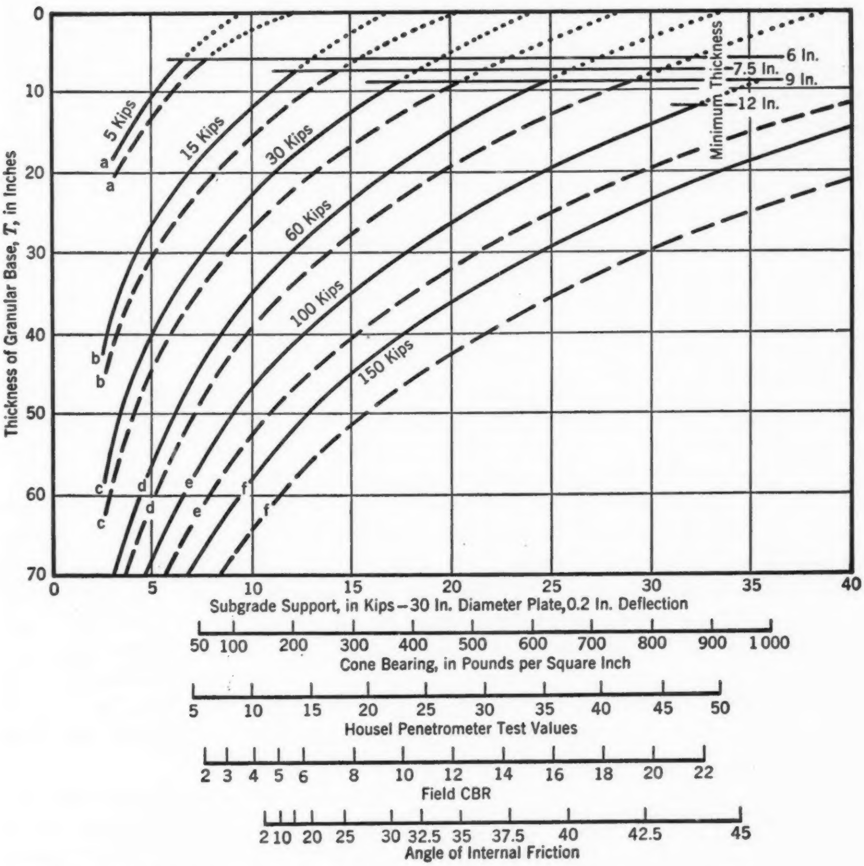

FIG. 17.—THICKNESS OF GRANULAR BASE, AS COMPARED WITH SUBGRADE SUPPORT

TABLE 4.—EXPLANATION OF CURVES IN FIG. 17

Pair of curves[a]	Wheel load, in kips	Contact pressure, in lb per sq in.	Base course constant K
a	5	45	35
b	15	55	51
c	30	65	59
d	60	75	67
e	100	100	69
f	150	125	72

[a] The solid lines are for runway design, and the dashed lines are for the design of taxiways, aprons, and turnarounds.

applied load, P, that produces a deflection of 0.5 in. at 10 repetitions of loading for runways and 0.35 in. at 10 repetitions for taxiways, and using the corresponding value of S. The value of the base course constant K varies with the size of the contact area, these areas being of approximately the size that might be expected for the wheel loads indicated. The curves of Fig. 17 apply to cohesive subgrade soils of fine texture. The values of cone bearing pressure, Housel penetrometer tests, field CBR, and the angle of internal friction, indicated in Fig. 17, represent the average rating for the top 2 ft of the subgrade. The Housel penetrometer test values are the number of blows for 6 in. of penetration. The subgrade support values to be used in entering Fig. 17 are those required for a plate having a 30-in. diameter, at a 0.2-in. deflection after 10 repetitions of loading.

Experience has demonstrated that a greater thickness of flexible pavement is required for taxiways, aprons, and the turnaround areas at runway ends than for the runways. From the data available, the Canadian Department of Transport has established a 0.35-in. deflection as the basic criterion for the design of taxiways, aprons, and turnarounds for capacity operations. The taxiways curves in Fig. 17 are based on this criterion.

For the design of flexible pavement runways, taxiways, aprons, and turnarounds, the following criteria have been established to govern the plate bearing tests: (1) In designing a runway for capacity operations, the criterion to be used is the load required to produce a 0.5-in. deflection after 10 repetitions; (2) in designing a runway for limited operation, the criterion is that load producing a 0.5-in. deflection after 1 repetition; (3) in designing taxiways, aprons, and turnarounds for capacity operations, the criterion is the load that produces a deflection of 0. 35 in. after 10 repetitions; and (4) the design of taxiways, aprons, and turnarounds for limited operations is based on the load producing a 0.35-in. deflection after 1 repetition of loading.

The use of the design charts can be illustrated by assuming that a capacity-operation runway is to be designed for a single-wheel load of 60 kips and a contact pressure of 75 lb per sq in. The subgrade support, measured at 0.2 in. of deflection with a bearing plate 30 in. in diameter is 10 kips. From Fig. 17 the required pavement thickness is 50 in. The same answer could have been arrived at by the use of Eq. 1.

Dual-wheel loadings are discussed by Mr. McLeod on the basis of his studies. From the limited data available, there is an indication that, for pavements from 6 in. thick to 10 in. thick, the dual-wheel assembly may carry up to 133% of the load on a single wheel without increasing the stress in the subgrade. Thus, a load of 60 kips on a dual-wheel assembly is equivalent to a single-wheel load of 45 kips. For pavements from 15 in. thick to 20 in. thick, the dual-wheel assembly may carry up to 125% of the single-wheel load.

The Navy Method

In determining the required thickness of flexible pavements, the Navy Department utilizes the theoretical approach developed by Donald M. Burmister,[7] A. M. ASCE. Mr. Burmister applies the theory of elasticity to the determination of pavement thickness.

[7] "Airfield Pavement," TP-Pw-4, Bureau of Yards and Docks, U. S. Navy, August, 1952.

The method requires the evaluation of the modulus of elasticity of the pavement, E_1, and of the subgrade, E_2. This is done by means of plate bearing tests on special test sections, using a steel plate 30 in. in diameter. The load capacity of the pavement is based on the criterion that the deflection (or settlement) under the wheel load must not exceed 0.2 in.

Basic Theory.—According to the theory of elasticity, the settlement under a loaded rigid circular plate (uniform settlement) resting on a homogeneous, elastically isotropic solid of infinite depth is

$$\Delta = \frac{w\,r}{2}\,\frac{1-\mu^2}{E_2} \quad \text{(2)}$$

in which Δ represents the settlement of the plate, in inches; w is the applied pressure, in pounds per square inch; r is the radius of the plate, in inches; μ stands for Poisson's ratio; and E_2 is the modulus of elasticity of the subgrade, in pounds per square inch.

If we assume that Poisson's ratio for most compacted soils is equal to 0.5, Eq. 2 becomes

$$\Delta = 1.18\,\frac{w\,r}{E_2} \quad \text{(3)}$$

According to the theory of elasticity, the settlement at the center of a loaded circular flexible plate that is uniformly loaded (equal reactions) is

$$\Delta = \frac{2\,w\,r(1-\mu^2)}{E_2} \quad \text{(4)}$$

in which w is equal to the intensity of loading, in pounds per square inch.

If 0.5 is substituted again for μ, Eq. 5 is obtained:

$$\Delta = \frac{1.5\,w\,r}{E_2} \quad \text{(5)}$$

When a pavement of thickness h and modulus of elasticity E_1 rests on a subgrade of infinite depth and modulus E_2, the two-layer system results.

Mr. Burmister extended the theory to the solution of two-layered systems by the introduction of a settlement coefficient F which is a multiplying (or correction) factor applied to Eq. 2 for the case of a homogeneous single layer. Eq. 3 then becomes

$$\Delta = 1.18\,\frac{w\,r}{E_2}\,F \quad \text{(6)}$$

and Eq. 4 becomes

$$\Delta = 1.5\,\frac{w\,r}{E_2}\,F \quad \text{(7)}$$

Application of the Theory.—The "settlement factor," F, is dependent on the ratio E_2/E_1, in which E_1 is the modulus of elasticity of the pavement in pounds per square inch. In Fig. 18, numerical values of F are plotted as ordinates.[7] Values of the ratio of the radius r, of the circular load to the pavement thickness, h_1 (for different values of E_2/E_1) are plotted as abscissas. These curves indicate the load-settlement characteristics for a two-layer soil system, and yield

values of settlement at the center of a flexible bearing area. It should be noted that, when the thickness h_1 is very great, r/h_1 approaches zero; and values of F become equal to the ratio, E_2/E_1.

As explained previously, E_2 (the subgrade modulus) is measured in the field by means of a plate bearing test utilizing a plate 30 in. in diameter. The test is made on the subgrade by placing on top of the 30-in.-diameter plate several plates of smaller diameters, giving considerable rigidity so as to simulate a rigid plate. In testing the subgrade (without pavement), Eq. 3 is used to evaluate E_2. However, in evaluating E_1 of the pavement, the rigid plate is placed on top of the pavement and Eq. 6 is used. It is important to recognize that Mr. Burmister's analysis was based on a uniformly-loaded circular flexible bearing area and not on a rigid plate. Therefore, in a strict sense, the nest of curves in Fig. 18 is not applicable to the latter case. However, applying them

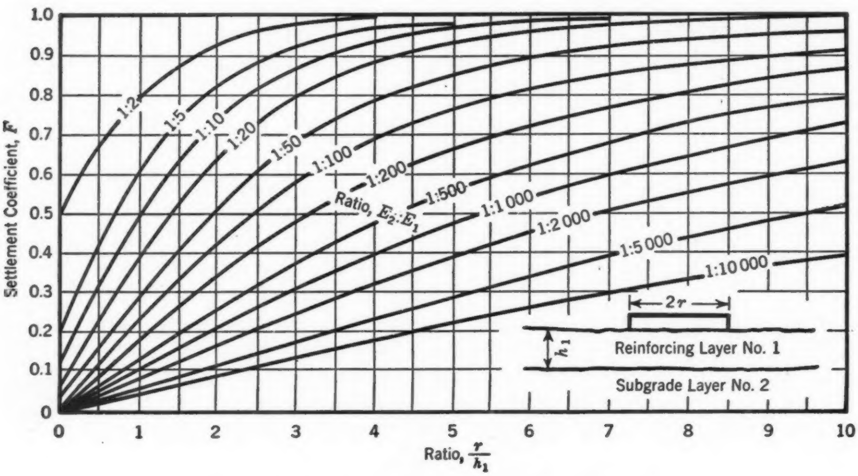

FIG. 18.—INFLUENCE CURVES FOR THE SETTLEMENT COEFFICIENT, F

to the rigid plate as the Navy method does introduces no serious error. After the physical constants, E_1 and E_2, are evaluated, the pavement thickness is determined from Eq. 7. The logic behind this is that a tire acts on a pavement (approximately uniform contact pressure) in a manner similar to that of the condition evaluated in Eq. 4.

The following example serves to illustrate the Navy procedure for designing flexible pavements. It is desired to determine the pavement thickness needed to support a single-wheel load of 50 kips. Assume that the contact pressure of the tire is 150 lb per sq in. The contact area is then $\frac{50,000}{150} = 333$ sq in. The radius of the equivalent circle whose area is 333 sq in. is 10 in. By preliminary plate loading tests on the subgrade with a plate 30 in. in diameter, it is found that on the average a load of 23 lb per sq in. is required to produce a plate settlement of 0.2 in. By substitution in Eq. 3, $E_2 = 1.18\, w\, r/\Delta$

= 2,030 lb per sq in. By a plate loading test on a preliminary test section of a base course 6 in. thick with a plate 30 in. in diameter, a load of 45 lb per sq in. causes a settlement of 0.2 in. By substitution in Eq. 6, $F = 0.510$. Entering Fig. 18 with 0.510 as the ordinate and $r/h_1 = 2.5$ as the abscissa, it is found that $E_2/E_1 = 1/100$ or $E_1 = 100 \times E_2 = 203{,}000$ lb per sq in. Substitution in Eq. 7 yields $F = \dfrac{2{,}030 \times 0.2}{1.5 \times 150 \times 10} = 0.17$. Entering Fig. 18 with $F = 0.17$ and $E_2/E_1 = 1/100$ yields a value for r/h_1 equal to 0.70. The theoretical thickness of the pavement, h_1, is then equal to 15 in.

Verification by Tests on Trial Sections.—After the preliminary determination of pavement thickness has been completed, it is verified by the construction of trial pavement sections in (*a*) fill sections, (*b*) cut sections, and (*c*) areas in which there is neither cut nor fill but in which only the removal of vegetal growth may be required. The trial pavement sections should not be less than 100 sq ft in area and the minimum dimension should not be less than 10 ft. Three trial sections are recommended for each of the categories of subgrade already mentioned: One having a pavement (without surfacing) of the estimated theoretical thickness; one having a thickness that is two thirds of the theoretical thickness; and a third section with a pavement thickness that is 1.5 times the theoretical thickness. In all, a minimum of nine trial sections is required. The trial sections are incorporated into the project, whenever feasible.

In order to compensate for a certain amount of subgrade moisture increase, subsequent to pavement construction, the following procedure is recommended when the subgrade is of a cohesive type. Samples of the top 1 ft of subgrade from fill, cut, and natural grade in which there is neither appreciable cut nor fill are tested for compaction characteristics (moisture-density relations) in the soils laboratory. Simple compression tests are then made on cylinders 2 in. in diameter and 4 in. high, cored from these subgrade samples. The samples are compacted (*a*) to 95% of the maximum density at optimum moisture content, as determined by a modified form of Test T99-49 of the American Association of State Highway Officials[8] (AASHO) and (*b*) to 95% of the maximum density that is obtainable by the modified AASHO method of compaction, when the soil moisture content is 2% higher than the optimum. The ratio of compressive strength for condition (*a*) to that for condition (*b*) is carefully determined from several tests and is applied as a correction factor to the plate loading test data. Thus, for a plate settlement of 0.2 in. on the subgrade of the trial section at condition (*a*), a settlement of 0.2 in. times the strength under condition (*a*) divided by the strength under condition (*b*) may be expected if the load on the pavement remains constant when the subgrade moisture increases from optimum to 2% more than optimum. The AASHO test is modified as follows: (1) Material is placed in the mold in approximately five equal layers; (2) the weight of the tamper is 10 lb instead of $5\frac{1}{2}$ lb; and (3) the tamper is dropped from a height of 18 in. instead of 12 in.

[8] "Standard Specifications for Highway Materials and Methods of Sampling and Testing, Part II," adopted by the Am. Assn. of State Highway Officials, Washington, D. C., 1950.

To complete the example, three trial fill sections are constructed having thicknesses of a compacted base course of 15 in., 10 in., and 22.5 in. A plate having a diameter of 21 in.—twice the radius (to the nearest inch) of the equivalent circle for a 50-kip tire load—is made for use in the tests on the base course. A plate 30 in. in diameter is used for tests on the subgrade. Assume that, in the example illustrating this procedure, the data of Table 5 were obtained.

The corrected settlements of line 5 in Table 5 are plotted against the pavement thicknesses as shown in Fig. 19.[7] A smooth curve is then drawn through those points corresponding to the greatest corrected settlements. The pavement thickness corresponding to a corrected settlement of 0.20 in. is determined from this curve. This value is seen to be 18 in. (to the nearest inch) and this is the pavement thickness selected. Since the asphalt surfacing

TABLE 5.—Test Data for Pavement Design, from Trial Pavement Sections

Procedure:
To obtain the values in line 3, divide the values in line 1 by the values in line 2. Line 5 contains the products obtained by multiplying the factors in line 3 by the values in line 4.

Line No.	Description	Test Section								
		Fill			Cut, Scarified, and Compacted			Natural Grade, Scarified and Compacted		
		Trial Pavement Thickness, h, in in.								
		15	10	22.5	15	10	22.5	15	10	22.5
	Average Compressive Strength of Laboratory Samples, in Lb per Sq In.:									
1	Optimum moisture	12	12	12	16.2	16.2	16.2	14.5	14.5	14.5
2	Optimum moisture plus 2%	8.4	8.4	8.4	14.0	14.0	14.0	9.8	9.8	9.8
3	Settlement correction factors	1.43	1.43	1.43	1.16	1.16	1.16	1.48	1.48	1.48
	Total Settlement y, in inches:									
4	Measured	0.163	0.236	0.120	0.190	0.274	0.130	0.135	0.203	0.084
5	Corrected	0.233	0.338	0.171	0.218	0.318	0.150	0.200	0.300	0.125

was omitted from the test sections and since the unit bearing contributed by the surfacing is usually greater than that of the base course, it is decided that, for a 3-in. surfacing, the thickness of the base course should be 15 in. which, together with the 3 in. of surfacing, will provide 18 in. of total pavement thickness.

The Design of Rigid Pavements, Using the Westergaard Method

Previous to World War II, the method for designing rigid pavements for airports was similar to that used for highway pavements. These design procedures were not applicable to the heavy loads and multiple-wheel landing gear developed during the war. To meet this situation, new methods of theoretical analysis were developed and several large programs of field testing of pavements were conducted.

On the theoretical side the late H. M. Westergaard, M., ASCE, expanded his previous work on highway pavements. He developed formulas of a much wider range of applicability with regard to the shape and size of loaded areas, and illustrated how they might be applied to the case of dual wheels making elliptical tire prints.[9,10] The Portland Cement Association (PCA), in cooperation with Kansas State College of Agriculture and Applied Science at Manhattan, further extended the Westergaard theory of pavement performance to make it applicable to any shape and size of loaded area produced by landing gear.[11,12] Design charts from which the required thickness of pavement may be read directly for various loads, types of landing gear, tire pressures, sub-

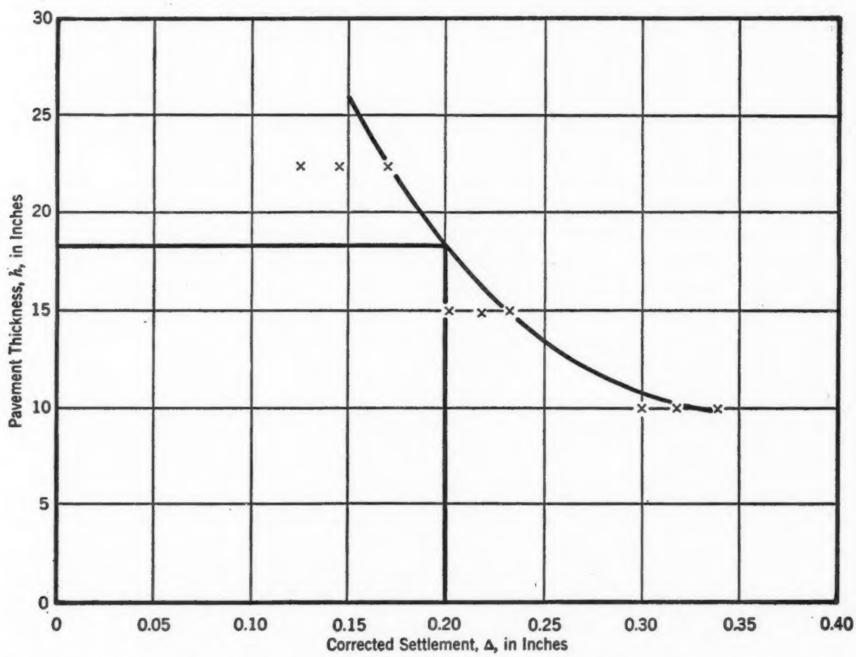

FIG. 19.—PAVEMENT THICKNESS, ON THE BASIS OF SUBGRADE SETTLEMENT UNDER LOAD

grade stiffnesses, and other influential factors are given in a design manual published by the PCA.[13]

Pavement testing at Wright Field, Dayton, Ohio; Hamilton Field, San Rafael, Calif.; Marietta, Ga.; Lockbourne, Ohio; and other locations by the

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[9] "Stress Concentrations in Plates Loaded over Small Areas," by H. M. Westergaard, *Transactions*, ASCE, Vol. 108, 1943, pp. 831–856.

[10] "New Formulas for Stresses in Concrete Pavements of Airfields," by H. M. Westergaard, *ibid.*, Vol. 113, 1948, pp. 425–444.

[11] "Influence Charts for Concrete Pavements," by Gerald Pickett and G. K. Ray, *ibid.*, Vol. 116, 1951, pp. 49–73.

[12] "Deflections, Moments and Reactive Pressures for Concrete Pavements," by Gerald Pickett, M. E. Raville, W. C. Jones, and F. J. McCormick, *Bulletin No. 65*, Kansas State College of Agri. & Applied Science, Manhattan, Kans., October, 1951.

[13] "Design of Concrete Airport Pavement," Portland Cement Assn., Chicago, Ill., 1950.

Corps of Engineers has fairly well substantiated the Westergaard theory of pavement performance as a workable basis for design. In general, measured stresses in the concrete are somewhat lower than those given by the theory; and therefore the theory leads to a conservative design—if other complicating factors do not interfere. A primary reason for the higher theoretical stresses is the fact that subgrades do not perform in the manner required by theory. A correction to the theory—to give reduced computed stresses—has been used

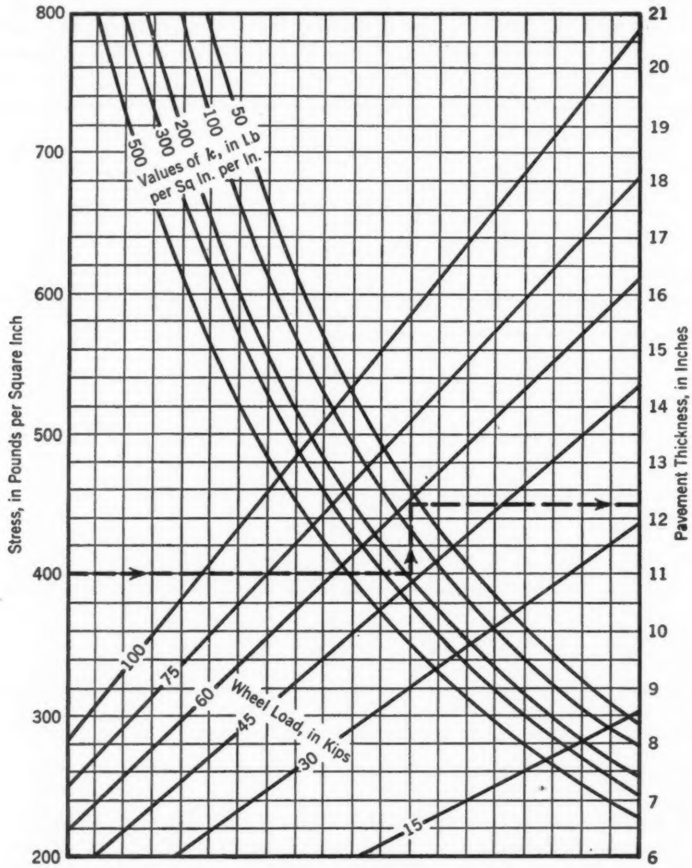

Fig. 20.—Rigid Pavement Design Chart, Single-Wheel Loading—Tire Pressure of 100 Lb per Sq In.

in highway design, but a similar correction has not appeared feasible for the shapes of loaded areas produced by large airplanes. An alternative theory—more in accord with subgrade performance and leading to a less conservative design—has been considered[12] but, so far as is known, has not been adopted by any agency in the United States.

The Westergaard theory involves a modulus of subgrade reaction, *k*, which is the ratio of the subgrade reaction to the subgrade deflection. The ratio

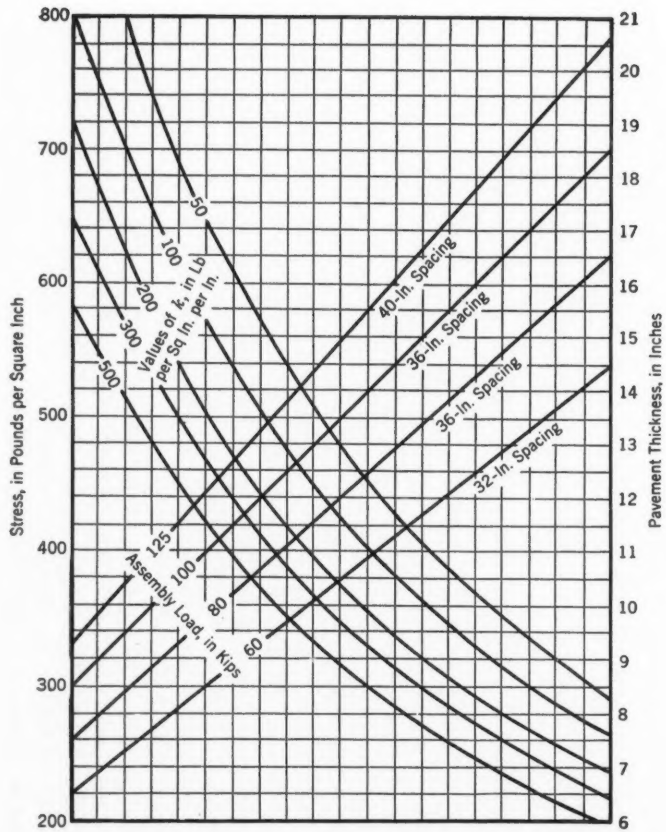

FIG. 21.—RIGID PAVEMENT DESIGN CHART, DUAL WHEELS SPACED AS SHOWN—TIRE PRESSURE 100 LB PER SQ IN.

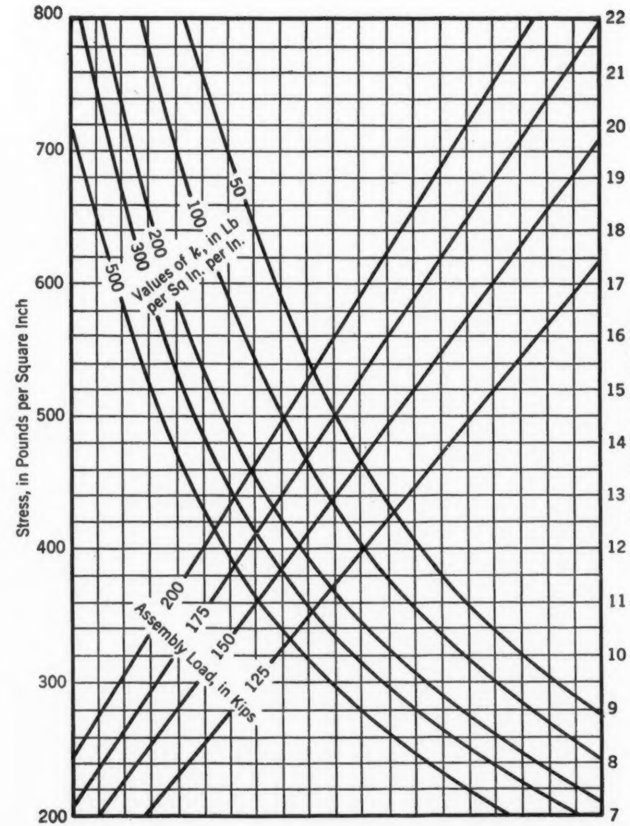

FIG. 22.—RIGID PAVEMENT DESIGN CHART, DUAL-TANDEM LANDING GEAR (WHEELS SPACED 31.5 IN. ABREAST AND 62 IN. TANDEM).

is determined by plate bearing tests on the subgrade. Thus, if an average load of 10 lb per sq in. is needed to cause a deflection of 0.05 in., the calculated value of the modulus of subgrade reaction would be $10/0.05 = 200$ lb per cu in. Because subgrades do not perform in the manner required by the theory, the value of k found for a given subgrade depends upon many factors, including the size of the plate used. The problem of determining k for use in design is the object of considerable study. Although other procedures are used, the most common method is based on the load needed to deflect a rigid plate 30 in. in diameter a distance of 0.05 in.

Figs. 20, 21, and 22, taken from a PCA design manual,[13] give the required thicknesses of concrete pavements for certain types of landing gears, based upon the Westergaard theory. Fig. 20 is a design chart for single-wheel loads of from 15 kips to 100 kips and for tire pressures of 100 lb per sq in. Fig. 21 is a design chart for dual-wheel assembly loads of from 60 kips to 125 kips and tire pressures of 100 lb per sq in. Fig. 22 is a design chart for dual-tandem assemblies for loads of from 125 kips to 200 kips and for a constant contact area of 267 sq in. per tire. All the design curves are based on a modulus of elasticity of 4,000,000 lb per sq in. and a Poisson ratio of 0.15. An increase in the Poisson ratio from 0.15 to 0.20 brings about an increase in stress of about 4%. A reduction in the modulus of elasticity of from 4,000,000 lb per sq in. to 3,000,000 lb per sq in. decreases the stress about 5%. The stresses resulting from a condition of loading other than that on which the design chart is based could be computed by the use of an influence chart.

To obtain the allowable stress it is customary to divide the modulus of rupture of the concrete by a factor of safety for which the PCA[13] suggests the values tabulated as follows:

Installation	Safety factor
Aprons, taxiways, hardstands, runway ends, or hangar floors	1.8 to 2.0
Runways (central section)	1.3 to 1.5

As an illustration of the use of the airport pavement design charts, a typical problem is presented. The thickness of runway pavement for a single-wheel load of 60 kips is required. The k-value of the subgrade is 200 lb per sq in., and the modulus of rupture of the concrete is 600 lb per sq in. The allowable stress of 400 lb per sq in. is obtained by dividing the modulus of rupture by the safety factor, 1.5. Enter the design chart of Fig. 20 on the left margin with a stress of 400 lb per sq in. and proceed horizontally to the right to the curved line for a k-value of 200 lb per sq in. per in. Proceed vertically to the diagonal line that represents a wheel load of 60 kips at a tire pressure of 100 lb per sq in. and then horizontally to the edge of the chart, where the required thickness of 12.2 in. is obtained.

The pavement thickness needed for dual-wheel landing gear of spacing other than that shown may be obtained directly without going to an influence chart, by applying the following approximate correction factors: (1) For increases in center-to-center tire spacing of up to 10 in., the required thickness of pavement should be reduced by 0.6% for each 1-in. increase in spacing; and (2) for each

1-in. decrease in dual-tire spacing, the required thickness should be increased by 0.6%. Correction factors of this type were not prepared for dual-tandem assemblies.

In concrete pavement, joints in the longitudinal and transverse directions must be provided to reduce the stresses caused by changes in the volume of the concrete. The spacing and types of longitudinal and transverse joints are adequately covered in other publications[13] and hence are not described herein.

Comparison of Design Procedures

The basic mission of the committee was to make quantitative comparisons of the thicknesses provided by the different methods of design. Although the name of the committee indicates that its principal objective was to correlate design procedures for runways, the word "correlation" could be replaced by "comparison" to better describe what was done. The correlation of empirical design procedures is not practicable, but a comparison of the end results, namely, of the pavement thicknesses, can be made.

Bases for Comparison.—One of the significant differences in all of the design procedures studied is the procedure for determining the "worst condition" of the subgrade during the life of the pavement. It is generally agreed that the moisture content and the state of compaction of a subgrade are both subject to change subsequent to the construction of a pavement, and that changes in either of these factors may involve a serious alteration in the stability of the pavement. Therefore, in designing pavements, it is essential to estimate the critical condition of moisture at which the supporting capacity of the subgrade should be determined.

The procedure used by the Corps of Engineers for permanent construction is based on the assumption that during the life of a pavement there will be an increase in moisture in the subgrade. It could become fully saturated, or nearly so. This is the principal reason for soaking a sample of subgrade material for four days prior to making the CBR test. However, it is understood that in recent years the Corps of Engineers has modified its procedure—by eliminating the soaking of laboratory samples of subgrade, using in-place tests, or reducing by 20% the pavement thickness that has been derived by the normal procedure of soaking the samples—for cases in which information has been available, indicating that the actual moisture conditions will not be as severe as the saturated condition. For cohesionless soils, soaking has very little or no effect on the CBR value. However, this is not the case for plastic soils, in which the reduction in CBR value may be considerable. One thing must be kept clearly in mind: The design curves for the CBR method are based on the correlations of accelerated traffic tests and airfield pavement service records with the in-place CBR value of the subgrade rather than with CBR values for soaked subgrades.

The Bureau of Yards and Docks made an investigation of the Navy airfields and found that the changes in moisture content were not significant and that there were few instances in which the maximum moisture content of the subgrade was above the plastic limit. However, the Navy design procedure for pavement thickness provides for a small gain in moisture.

In contrast to the CBR (Corps of Engineers) method and the Navy method, the design charts of the Canadian Department of Transport are based on the determinations of subgrade strength made on runways that had been paved for several years. The Canadian Department of Transport believes that these runways have been in service for a sufficient period of time to allow the moisture content of the subgrade to arrive at its equilibrium value, and that therefore no further reduction in strength because of moisture accumulation is indicated. Therefore, the subgrade support measured in the field is used without modification in this design method.

The CAA does not use a strength index test. The pavement thickness has been correlated with a subgrade rating that includes the effect of local climatic and moisture conditions on the stability of the subgrade soil.

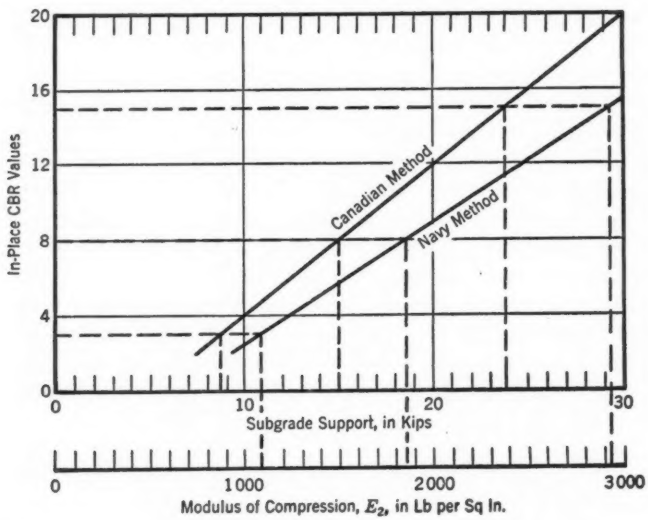

FIG. 23.—COMPARISON OF IN-PLACE CBR WITH THE SUBGRADE SUPPORT AND E_2

Thus, the basis for comparison must be fully understood in order to appreciate properly a comparison of thicknesses. One possible comparison is based on the assumption that the characteristics and condition—and hence strength—of the subgrade were substantially the same for all methods. In effect, this condition excludes any consideration of future reduction in subgrade supporting capacity because of moisture and predicates the design entirely on the subgrade condition at the time of testing. Because of this exclusion, it should be emphasized that the procedures followed in evaluating the thicknesses for this comparison deviate from those normally used by the various agencies in design work.

Comparisons of Flexible Pavement Design Methods.—The first comparison of flexible pavement thicknesses was made on the basis of the assumption that the subgrade conditions were approximately the same for each design procedure and that no change in these conditions was expected during the life of the

pavement. An attempt has been made to compare the thicknesses derived by the CBR method, the Canadian method, and the Navy method, utilizing this assumption. The CAA method was not included in this comparison because it does not use a direct strength measurement in the field; consequently, there was no way to evaluate thicknesses on the basis of in-place conditions. The curves in Fig. 23 show the relationship between the in-place CBR values, the subgrade support values of the Canadian method, and the modulus of compression, E_2, used in the Navy Method. The scale of abscissas represents the subgrade support beneath a plate 30 in. in diameter at a deflection of 0.2 in. The ordinate values are CBR values corresponding to a 0.1-in. penetration. The curve for the Canadian method has appeared previously.[6] The moduli of compression, E_2 (Navy method) were computed on the basis of the data from Canadian tests using Eq. 3. Thus, a subgrade support of 10 kips beneath a plate 30 in. in diameter corresponds to

$$E_2 = \frac{1.18 \dfrac{10{,}000}{\text{area of plate}} (15)}{0.2} = 1{,}250 \text{ lb per sq in.}$$

An attempt was made to compare, in so far as possible, the runway thicknesses obtained by the three methods of design for three different subgrade conditions, considering a single wheel with a contact pressure of 100 lb per sq in. The three subgrade conditions selected for comparison had CBR ratings of 3, 8, and 15. A CBR of 3 represents a poor subgrade of low bearing capacity; a CBR of 8 represents a somewhat better subgrade; and a CBR of 15 represents a medium strength subgrade. Utilizing Fig. 23 and the CBR method (Corps of Engineers), the Navy method, and the Canadian method, pavement thicknesses for these three different subgrade conditions are shown in Figs. 24(*a*), 24(*b*), and 24(*c*). In computing the thickness of pavement by the Navy method, the modulus E_1 of the pavement was assumed as 200,000 lb per sq in. L. A. Palmer[14] has stated that

> "The measured moduli of base courses tend to vary over a wide range, those for stabilized aggregate base course being generally between 25,000 and 100,000 lb per sq in. Those for well-constructed penetration macadam, black base, and water-bound macadam tend usually to exceed 100,000 lb per sq in. and in some instances are as high as 200,000 lb per sq in."

The flexible pavement structure in this example was assumed to be made up of from 2 in. to 3 in. of asphaltic concrete, from 6 in. to 10 in. of high quality base, and, for the larger wheel loads, a layer of selected material. It was felt that a value of 200,000 lb per sq in. was a reasonable assumption as to E_1 for the pavement as a whole. If this value was reduced to 100,000 lb per sq in., the thicknesses determined by the Navy method would be about 25% greater than those shown in Fig. 24. It is emphasized that the thicknesses used in design by the Navy method are based on test sections. Theory is used primarily for establishing the thickness of the section. The final design thicknesses are usually larger than those computed from theory alone. The committee used the theoretical thicknesses for the comparisons because there were then no relation-

[14] "The Pavement Extension and Reinforcement Program of the Bureau of Yards and Docks," by L. A. Palmer, *Proceedings-Separate*, ASCE (publication pending).

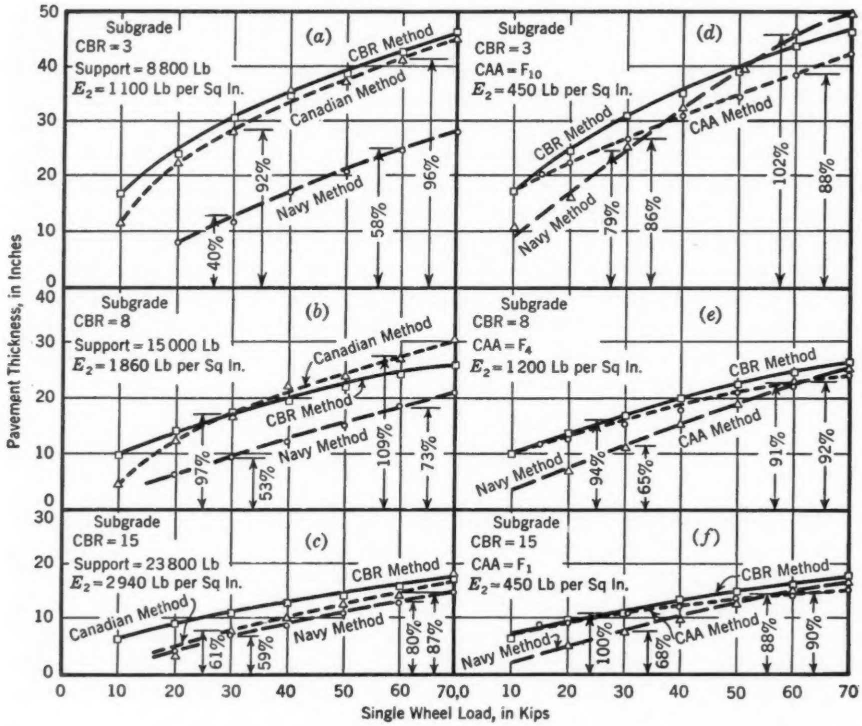

FIG. 24.—COMPARISON OF THE PAVEMENT THICKNESSES DETERMINED BY FOUR DIFFERENT METHODS

TABLE 6.—DATA AND RESULTS OF THE COMPARISONS IN FIG. 24

Set of curves in Fig. 24	PAVEMENT THICKNESSES, AS % OF THE RESULTS OBTAINED BY THE CBR METHOD — SINGLE-WHEEL LOADS, IN KIPS: 30			60		
	Canadian method	CAA method	Navy method	Canadian method	CAA method	Navy method
(1)	(2)	(3)	(4)	(5)	(6)	(7)
COMPARISON BASED ON IN-PLACE TESTS						
(a)	92	...	40	96	...	58
(b)	97	...	53	109	...	73
(c)	61	...	59	87	...	80
COMPARISON BASED ON ENVIRONMENTAL EFFECTS FROM MOISTURE						
(d)	...	86	79	...	88	102
(e)	...	94	65	...	92	91
(*f*)	...	100	68	...	88	90

ships available between the theoretical and design thicknesses. An examination of the few records available indicates that the difference is not large.

For wheel loads of 30 kips and 60 kips, the pavement thicknesses determined by the Canadian method and the Navy method are compared with those determined by the CBR (Corps of Engineers) method, the data coming from Fig. 24. This comparison comprises Cols. 2 through 7, Table 6. The following facts are indicated by Table 6:

1. As the bearing capacity of the subgrade increases, the difference in thickness between the results of the Navy method and those of the CBR method decreases. (Read down Col. 4.)

2. For the same subgrade, the difference in thickness between the results of the Navy method and those of the CBR method decreases as the wheel load increases. (Compare Col. 4 with Col. 7.)

3. For the weak and moderately weak subgrades (CBR values of 3 and 8), the thicknesses of the Canadian method are nearly the same as the corresponding thicknesses of the CBR method.

4. For the stronger subgrade (CBR 15), the thicknesses of the Canadian method are considerably smaller than the corresponding thicknesses determined by CBR method and more nearly approach those of the Navy method.

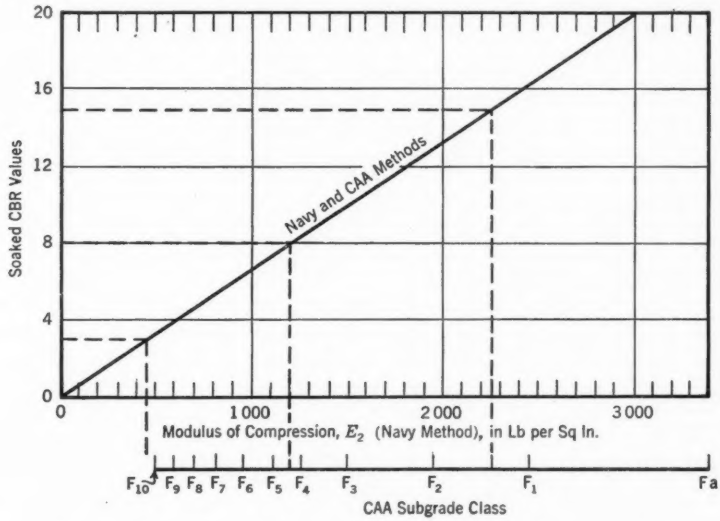

FIG. 25.—COMPARISON OF SOAKED CBR WITH E_2 AND CAA CLASSIFICATION

The second comparison is based on the assumption that the thicknesses are those required when the subgrade is in its "worst" condition. As explained previously, the procedure for creating a "worst" condition varies according to the design method adopted; hence, the comparison is based on differing subgrade conditions.

Fig. 25 shows the relationship between CBR values, and CAA subgrade class F_a, F_1, F_2, F_3, etc., and the modulus of compression E_2 of the Navy

method. The CBR values used as ordinates for this comparison were those of soaked subgrade material. The Canadian method has not been included. In order to include it in the comparison, the ratio of CBR values for in-place tests to those of soaked samples must be known because the only correlation available between results of the Canadian method and the CBR method is on the basis of in-place tests. A reasonable relationship between CBR values for in-place and soaked subgrade material has not been developed. The ratio of the results of in-place CBR tests to those for soaked samples is a function of soil type, density, initial water content, and other related variables. The Canadian Department of Transport has found this ratio to range all the way from 1 (or slightly less) to 10. For a cohesionless sand or gravel the ratio would approach unity, and would become larger for fine compressible soils such as clays.

Utilizing Fig. 25 and the design procedures previously described, pavement thicknesses for different subgrade conditions (CBR values of 3, 8, and 15) are shown in Figs. 24(*d*), 24(*e*), and 24(*f*). The information presented in this figure is summarized in Table 6, which indicates the following:

a. The thicknesses derived by the CAA method agree fairly well with those of the CBR method. (Compare with Fig. 24.)

b. The thicknesses derived by the Navy method are less than those of the CBR method for the 30-kip load and agree fairly well for the 60-kip load.

Comparison of Rigid Pavement Design Methods.—So far, the presentation has been concerned with flexible pavements. All the airport agencies with the exception of the CAA use the Westergaard theory for designing Portland cement concrete pavements. An attempt was made to compare the thicknesses of slab by the CAA method with the thicknesses determined by the Westergaard analysis.

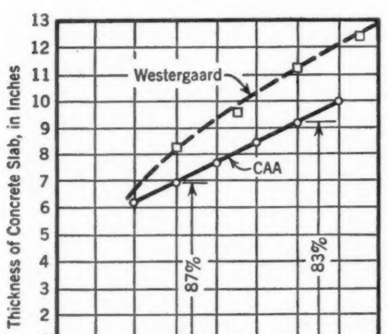

FIG. 26.—COMPARISON OF RIGID PAVEMENT THICKNESSES

The CAA procedure requires that a base of granular material of variable thicknesses (depending on the characteristics of the subgrade) be placed on the subgrade prior to placement of the slab. The thickness of the slab is dependent primarily on the wheel load because the bearing capacity of the foundation for the slab is kept almost constant by varying the thickness of the base. In effect, therefore, the base improves the bearing capacity of the subgrade. The CAA curves for concrete pavements can only be compared with other design curves on the basis that variable thicknesses of subbase will produce a constant modulus of subgrade reaction, k, of the order of 300 lb per cu in. Accordingly, the comparison with the Westergaard analysis was made, as shown in Fig. 26, on the basis of a k-value of 300 lb per cu in. As in the case of the flexible

pavement, the comparison was based on a single-wheel load with a contact pressure of 100 lb per sq in. The thicknesses according to the Westergaard method were obtained from a design chart.[13] The twenty-eight-day ultimate flexural strength of the concrete was assumed as 650 lb per sq in. and the factor of safety as 1.5. A value of 4×10^6 lb per sq in. was used for E, and μ was assumed to be 0.15. The curve of CAA thicknesses in Fig. 26 corresponds to values given by the R_1 curve of Fig. 3. An examination of Fig. 26 reveals that the CAA thicknesses vary from 83% to 87% of the Westergaard thickness, the difference increasing as the wheel load increases.

Conclusion.—The comparisons presented in Fig. 24 are only general trends of a qualitative nature and should not be construed as precise indications of the differences in the various design procedures. Figs. 23 and 25 yield values of the subgrade strength index (E_2, F_1, or subgrade support) for a particular design method corresponding to a certain CBR. These values are the key to the formulation of comparisons because they are used in the appropriate design procedures to solve for the thickness values plotted in Fig. 24. The comparison of design procedures purposely omitted any reference to the action of frost. Frost action is a complex subject and its inclusion in a quantitative comparison of thickness was beyond the scope of the committee's work.

Acknowledgements

The committee is grateful to Messrs. Gayle McFadden, T. B. Pringle, W. J. Turnbull, Members, ASCE, and C. M. Foster of the Corps of Engineers, for their generosity in supplying information for this report. Figs. 8, 9, 10, 11, and 12, appear through the courtesy of the Corps of Engineers. Mr. Palmer,[14] of the Bureau of Yards and Docks, supplied the data on the soaked CBR as compared with the modulus of compression, E_2, in Fig. 25. Fig. 18 also appears through the courtesy of the Bureau of Yards and Docks. Mr. McLeod very generously furnished information, including the data for correlation of in-place CBR and subgrade support in Fig. 23. Figs. 13, 15, and 16 appear through the courtesy of Mr. McLeod. Figs. 2, 3, and 7 appear through the courtesy of the CAA, who also supplied the data for comparing the CBR with the CAA subgrade classes in Fig. 23.

Respectfully submitted,

Henry Aaron — Robert Horonjeff
Harold Allen — Gerald Pickett

Walter Johannessen, *Chairman*

Committee on Correlation of Runway Design Procedures of the Air Transport Division

September 11, 1952